

CONFIDENTIAL

WOODRIDGE LAKE DAM

OT 00452



DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
424 TRAPELO ROAD
WALTHAM, MASSACHUSETTS 02154

REPLY TO
ATTENTION OF:

NEDED

NOV 28 1979

Honorable Ella T. Grasso
Governor of the State of Connecticut
State Capitol
Hartford, Connecticut 06115

Dear Governor Grasso:

Inclosed is a copy of the Woodridge Lake Dam Phase I Inspection Report, which was prepared under the National Program for Inspection of Non-Federal Dams. This report is presented for your use and is based upon a visual inspection, a review of the past performance and a brief hydrological study of the dam. A brief assessment is included at the beginning of the report. I have approved the report and support the findings and recommendations described in Section 7 and ask that you keep me informed of the actions taken to implement them. This follow-up action is a vitally important part of this program.

A copy of this report has been forwarded to the Department of Environmental Protection, the cooperating agency for the State of Connecticut. In addition, a copy of the report has also been furnished the owner, Woodridge Lake Property Owner's Association, Goshen, Connecticut 06757.

Copies of this report will be made available to the public, upon request, by this office under the Freedom of Information Act. In the case of this report the release date will be thirty days from the date of this letter.

I wish to take this opportunity to thank you and the Department of Environmental Protection for your cooperation in carrying out this program.

Sincerely,

MAX B. SCHEIDER
Colonel, Corps of Engineers
Division Engineer

Incl
As stated

HOUSATONIC RIVER BASIN
GOSHEN, CONNECTICUT

WOODRIDGE LAKE DAM CT 00452

PHASE I INSPECTION REPORT NATIONAL DAM INSPECTION PROGRAM



DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASS. 02154

AUGUST, 1979

BRIEF ASSESSMENT

PHASE I INSPECTION REPORT

NATIONAL PROGRAM OF INSPECTION OF DAMS

Name of Dam:	WOODRIDGE LAKE DAM
Inventory Number:	CT 00452
State Located:	CONNECTICUT
County Located:	LITCHFIELD
Town Location:	GOSHEN
Stream:	MARSHEPAUG RIVER
Owner:	WOODRIDGE LAKE PROPERTY OWNER'S ASSOCIATION
Date of Inspection:	MAY 3, 1979
Inspection Team:	PETER HEYNEN, P.E. CALVIN GOLDSMITH MIRON PETROVSKY GEORGE STEPHENS JAY COSTELLO


The dam, substantially completed in early 1970, is an earthfill embankment with a concrete spillway and is based on a till foundation. The embankment is 1320 feet in length, 34 feet in height, and 14 feet wide at the crest. The upstream slope inclination is 2 horizontal to 1 vertical and the downstream slope is 3 horizontal to 1 vertical. The spillway consists of an 80 foot long concrete ogee weir and a 40 foot wide by 144 foot long rectangular concrete chute with an energy dissipater and stilling basin. The outlet works consist of mid-depth and low-level concrete intake structures, a concrete valve chamber, 24 inch diameter drain pipe and a concrete low level outlet structure. The gate valves of the drain pipe and mid-depth steel intake pipe are operable.

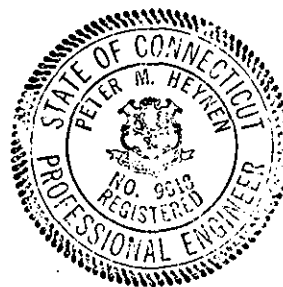
Based upon the visual inspection at the site and past performance of the dam, the dam is judged to be in good condition. No evidence of instability of the embankment or appurtenant structures was observed. There are some areas requiring monitoring and minor maintenance, such as the swamp at the right side of the dam toe and a rehabilitation of piezometers.


In accordance with Corps of Engineers Guidelines and the size (Intermediate) and hazard (High) classification of the dam, the test flood will be equivalent to the Probable Maximum Flood (PMF). Peak inflow to the lake is 12,600 cfs; peak outflow is 7,850 cfs with the dam overtopped 0.3 feet. The spillway capacity is 6340 cubic feet per second (cfs), which is equivalent to 81% of the routed test flood outflow.

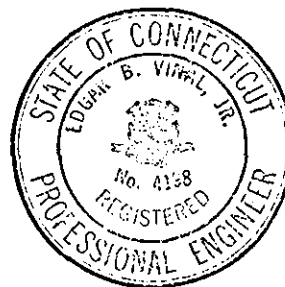
Further studies should be conducted to identify the origin of the extensive wet area at the toe of the embankment. The damaged piezometers should be repaired and a regular program for monitoring of the seepage and the dam drainage system should be established.

The above recommendations and any further remedial measures which are discussed in Section 7, should be instituted within two years of the owner's receipt of this report.

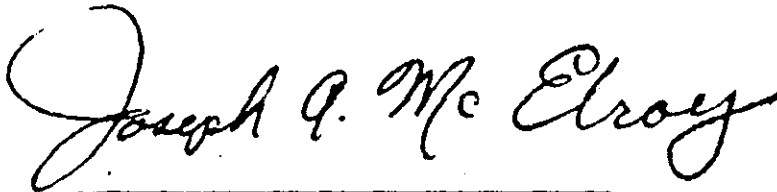

Peter M. Heynen, P.E.
Project Manager
Cahn Engineers, Inc.



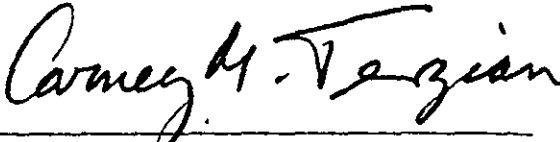

Edgar B. Vinal, Jr., P.E.
Senior Vice President
Cahn Engineers, Inc.



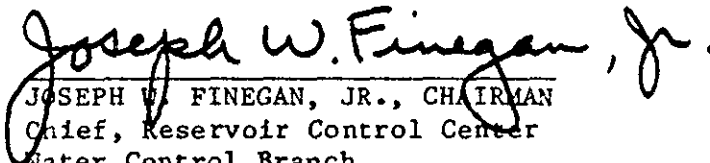
This Phase I Inspection Report on Woodridge Lake Dam has been reviewed by the undersigned Review Board members. In our opinion, the reported findings, conclusions, and recommendations are consistent with the Recommended Guidelines for Safety Inspection of Dams, and with good engineering judgment and practice, and is hereby submitted for approval.



JOSEPH A. MCELROY, MEMBER
Foundation & Materials Branch
Engineering Division



CARNEY M. TERZIAN, MEMBER
Design Branch
Engineering Division



JOSEPH W. FINEGAN, JR., CHAIRMAN
Chief, Reservoir Control Center
Water Control Branch
Engineering Division

APPROVAL RECOMMENDED:



JOE B. FRYAR
Chief, Engineering Division

PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams, for Phase I Investigations. Copies of these guidelines may be obtained from the Office of Chief of Engineers, Washington, D.C. 20314. The purpose of a Phase I Investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspection. Detailed investigation, and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I Investigation; however, the investigation is intended to identify any need for such studies.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data available to the inspection team. In cases where the reservoir was lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam would necessarily represent the condition of the dam at some point in the future. Only through continued care and inspection can there be any chance that unsafe conditions will be detected.

Phase I inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established Guidelines, the Spillway Test Flood is based on the estimated "Probable Maximum Flood" for the region (greatest reasonably possible storm runoff), or fractions thereof. Because of the magnitude and rarity of such a storm event, a finding that a spillway will not pass the test flood should not be interpreted as necessarily posing a highly inadequate condition. The test flood provides a measure of relative spillway capacity and serves as an aid in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition and the downstream damage potential.

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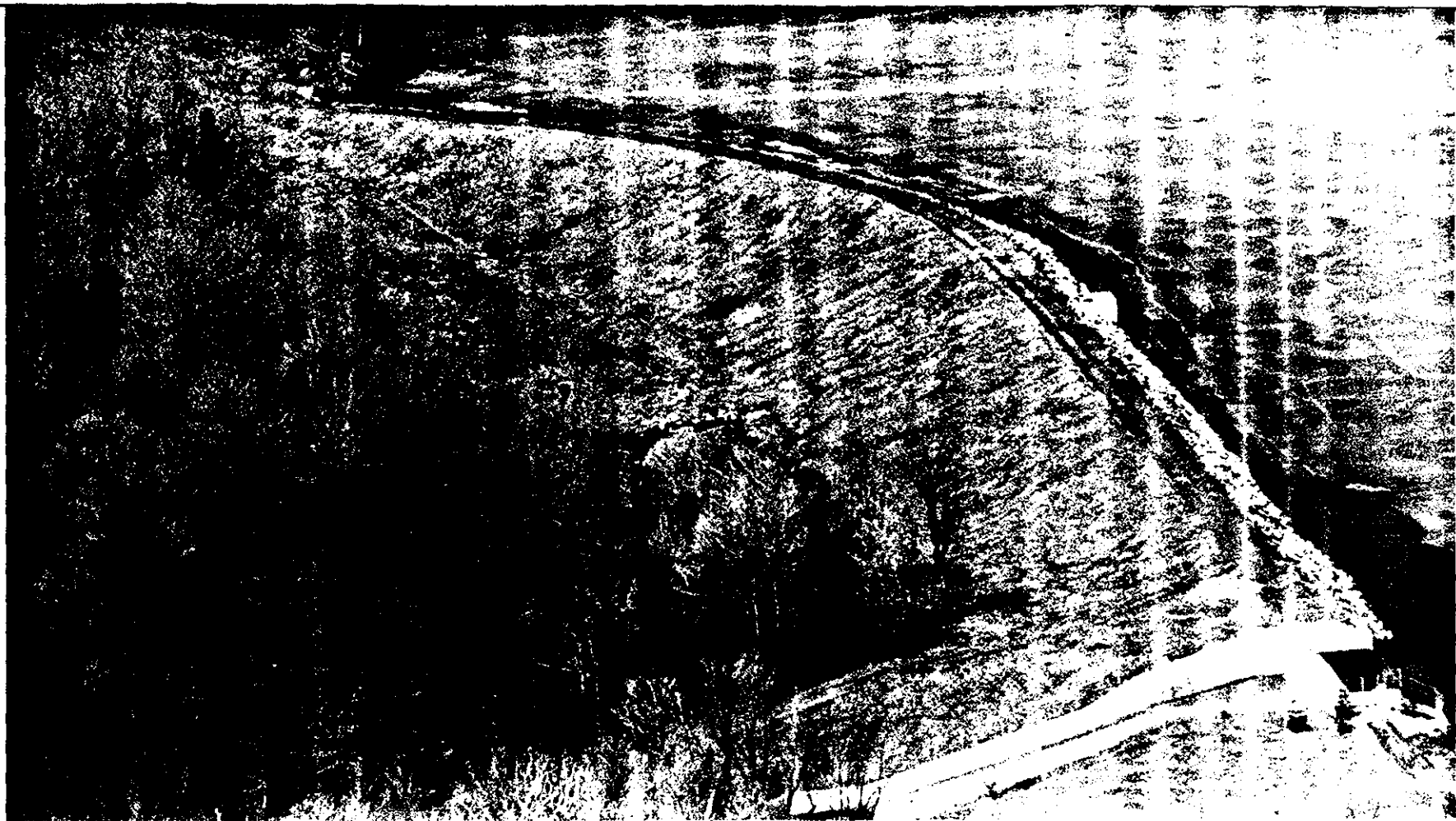
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OVERVIEW PHOTO

US ARMY ENGINEER DIV. NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS.

CAHN ENGINEERS INC.
WALLINGFORD, CONN.
ENGINEER

NATIONAL PROGRAM OF
INSPECTION OF
NON-FED DAMS

WOODRIDGE LAKE DAM

MARSHEPAUG RIVER

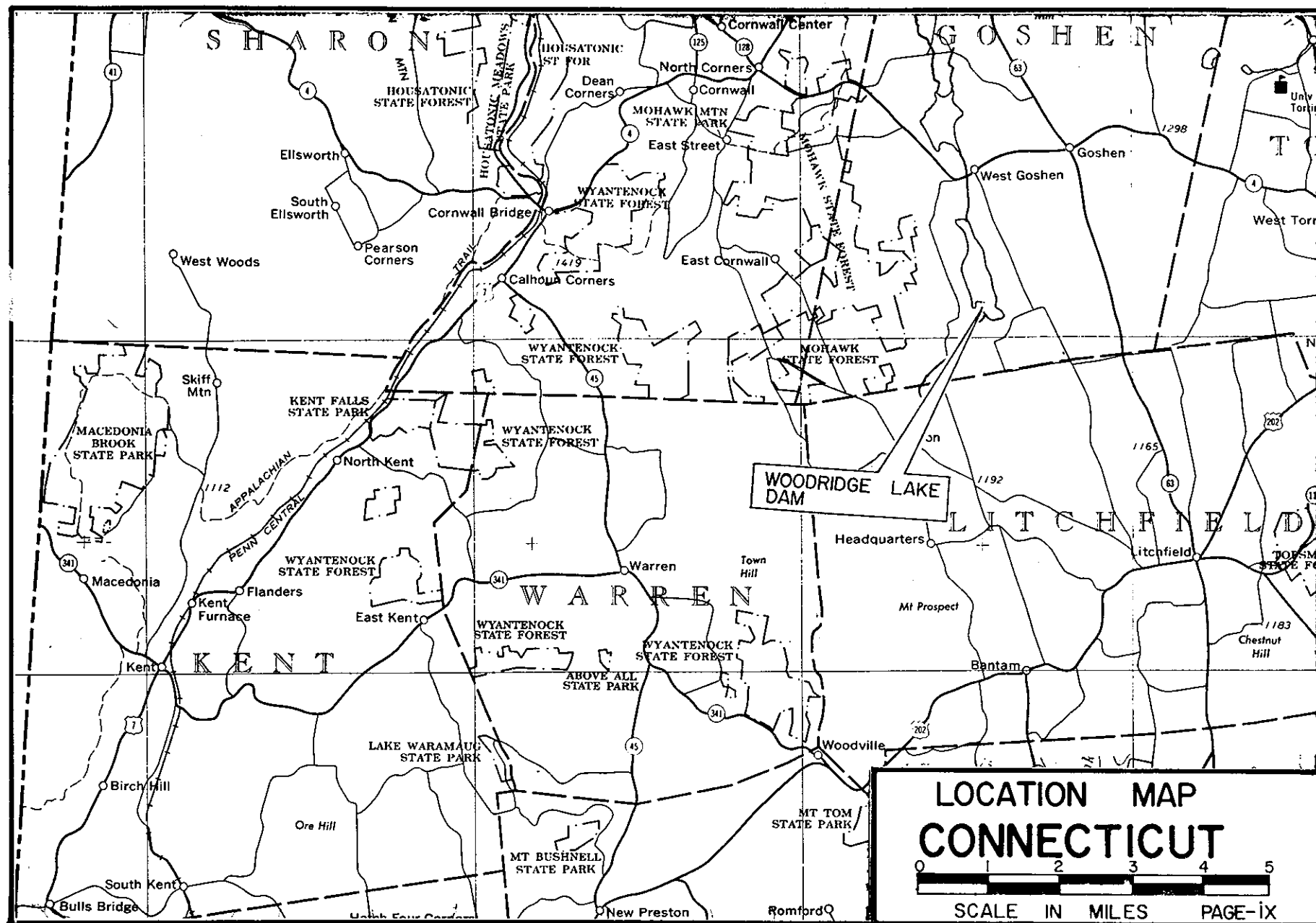
GOSHEN

CONNECTICUT

DATE March '79

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PAGE viii



PHASE I INSPECTION REPORT

WOODRIDGE LAKE DAM

SECTION I - PROJECT INFORMATION

1.1 GENERAL

a. Authority - Public Law 92-367, August 8, 1972, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a National Program of Dam Inspection throughout the United States. The New England Division of the Corps of Engineers has been assigned the responsibility of supervising the inspection of dams within the New England Region. Cahn Engineers, Inc. has been retained by the New England Division to inspect and report on selected dams in the State of Connecticut. Authorization and notice to proceed were issued to Cahn Engineers, Inc. under a letter of March 30, 1979 from John P. Chandler Colonel, Corps of Engineers. Contract No. DACW 33-79-C-0059 has been assigned by the Corps of Engineers for this work.

b. Purpose of Inspection Program - The purposes of the program are to:

1. Perform technical inspection and evaluation of non-federal dams to identify conditions requiring correction in a timely manner by non-federal interests.
2. Encourage and prepare the States to quickly initiate effective dam inspection programs for non-federal dam.
3. To update, verify and complete the National Inventory of Dams.

c. Scope of Inspection Program - The scope of this Phase I inspection report includes:

1. Gathering, reviewing and presenting all available data as can be obtained from the owners, previous owners, the state and other associated parties.
2. A field inspection of the facility detailing the visual condition of the dam, embankments and appurtenant structures.
3. Computations concerning the hydraulics and hydrology of the facility and its relationship to the calculated flood through the existing spillway.

4. An assessment of the condition of the facility and corrective measures required.

It should be noted that this report does not pass judgment on the safety or stability of the dam other than on a visual basis. The inspection is to identify those features of the dam which need corrective action and/or further study.

1.2 DESCRIPTION OF PROJECT

a. Location - The dam is located on the Marshepaug river in a rural area of the town of Goshen, County of Litchfield, State of Connecticut. The dam is shown on the Cornwall USGS Quadrangle Map having coordinates latitude N 41° 47.8' and longitude W 73° 15.1'.

b. Description of Dam and Appurtenances - The dam, completed in 1970, consists of a rolled earthfill embankment having a total length of approximately 1320 feet, an 80 foot long concrete spillway at the left side of the dam, and outlet works at the central portion of the dam.

The glacial till embankment has a top elevation of 1148.0, is 34 feet in height above the streambed and is 14 feet wide at the crest. A 9 inch thick gravel road is used as a cover for the dam crest. The upstream slope, inclined at 2 horizontal to 1 vertical, has 18 inch thick dumped rock riprap based on a 12 inch thick gravel bedding between elevation 1135 and the crest. The upstream toe of the dam has an impervious blanket, 200 feet wide and 3 feet thick, connecting with the upstream slope (See sheet B-1). The downstream slope, inclined at 3 horizontal to 1 vertical, is covered with 6 inch to 10 inch thick seeded topsoil. Under the topsoil, from elevation 1135 and down the slope, there is an 18 inch thick gravel drainage blanket joining with the gravel toe drain. This drain consists of a longitudinal 12 inch perforated metal pipe, 3 foot diameter vertical pressure relief wells on a 50 foot spacing and a 12 inch diameter collector drainage well. Also, along the downstream toe, there is a 4 foot wide stone surface drain leading to the outlet diversion channel.

The concrete spillway has a crest elevation of 1140.0 and a total length of 211 feet consisting of an uncontrolled ogee weir 5 feet in height and 80 feet in length, a 43 foot long transition section and a 40 foot wide by 144 foot long chute with an energy dissipator and stilling basin. The spillway is founded on a 20 inch thick crushed stone and gravel bedding. A gated concrete chamber and fishway are incorporated into the left side of the spillway.

The outlet works are mid-depth and low-level concrete intake structures for 6 inch and 24 inch pipes respectively, a concrete valve chamber with 6 inch and 24 inch gate valves, a 24 inch ductile iron drain pipe (at invert elevation 1116.0) from the valve chamber, and a concrete outlet structure. All outlets are operable.

Instrumentation of the earth embankment consists of 4 vertical open-system piezometers at the central part of the dam and a vee-notch weir for measurement of seepage from the toe drain (Sheet B-1).

c. Size Classification - INTERMEDIATE - The dam impounds 9800 acre-feet of water with the lake level at the top of the dam, which at elevation 1148.0, is 34 feet above the original streambed. According to the Recommended Guidelines, the dam is classified as intermediate in size.

d. Hazard Classification - HIGH - If the dam was to be breached, there is potential for loss of life and extensive property damage. At approximately 1/4 of a mile downstream of the dam on the Marshepaug River there are 7 residential structures 10 to 12 feet above the streambed. With a rapid rise in flood stage from 8.5 feet to 20.5 feet, these homes would be jeopardized upon failure of the dam.

e. Ownership - Woodridge Lake Property Owner's Association
Box 11
Goshen, CT 06756
Mr. William Donaldson, President
(203) 491-3424

Preliminary correspondence concerning dam construction was first begun by the West Goshen Realty Association, Inc. in 1964. Before the dam was constructed however, all property and plans for the dam were sold to Boise Cascade Properties Inc. in 1969. The dam was then completed in 1972 and has since been acquired by the present owners.

f. Operator - Mr. Tulli Amicone
Tel: (203)-491-3424
(203)-482-1582 (home)

g. Purpose of Dam - Recreation

h. Design and Construction History - The following information is believed to be accurate based on the plans and correspondence available. The dam was originally designed by Anderson-Nichols and Company, Inc. in 1966. Before construction was started however, the design was contracted to and revised by E. D'Appolonia Consulting Engineers, Inc. for Boise Cascade Properties Inc.

i. Normal Operational Procedures - Valves are operated during summer months to regulate the lake level or during extremely low flows when the 6 inch low flow augmentation system is opened to meet minimum downstream flow requirements. This low flow system has a design capacity of 2.5 cfs, which was established in accordance with a request by the City of Waterbury and the normal flow of 2 cfs at gaging station number 2019.3, located on the Marshepaug River 500 feet downstream from the dam. The lake elevation is dropped 5 feet during winter months but is normally maintained at 1140.

1.3 PERTINENT DATA

a. Drainage Area - 8.9 square miles of moderately steep, relatively undeveloped terrain which is 40% open and 60% wooded.

b. Discharge at Damsite - Discharge is from over the spillway and through the 24 inch low level outlet and 6 inch low-flow system, both of which are operated from the valve chamber.

1. Outlet works (conduits):

One 24" ductile iron pipe @ Invert El. 1116	70 cfs
---	--------

One 6" steel pipe @ Invert El. 1128+	2.5 cfs
--------------------------------------	---------

2. Maximum known flood at damsite:	N/A
------------------------------------	-----

3. Ungated spillway capacity @ top of dam el. 1148.0:	6340 cfs
---	----------

4. Ungated spillway capacity @ test flood el.:	6700 cfs
--	----------

5. Gated spillway capacity @ normal pool el.:	N/A
---	-----

6. Gated spillway capacity @ test flood el.:	N/A
--	-----

7. Total spillway capacity @ test flood el.:	6700 cfs
--	----------

8. Total project discharge @ test flood el. 1148.3:	7850 cfs
---	----------

c. Elevations (Feet Above Mean Sea Level)

1. Streambed at centerline of dam:	1114±
2. Maximum tailwater:	N/A
3. Upstream portal invert diversion tunnel:	N/A
4. Recreation pool:	1140±
5. Full flood control pool:	N/A
6. Spillway crest (ungated):	1140
7. Design surcharge (original design):	1144.5
8. Top of dam:	1148
9. Test flood surcharge:	1148.3

d. Reservoir

1. Length of maximum pool:	8200 ft.
2. Length of recreation pool:	7500 ft.
3. Length of flood control pool:	N/A

e. Storage

1. Recreation pool:	6500 acre-ft.
2. Flood control pool:	N/A acre-ft.
3. Spillway crest pool:	6500 acre-ft
4. Top of dam	9800 acre-ft.
5. Test flood pool:	9800 acre-ft.

f. Reservoir Surface

1. Recreation pool:	385 acres
2. Flood control pool:	N/A
3. Spillway crest:	385 acres
4. Top of dam:	430 acres
5. Test flood pool:	430 acres

g. Dam

- | | |
|---------------------|--|
| 1. Type: | Earthfill embankment |
| 2. Length: | 1320 [±] ft. |
| 3. Height: | 34 [±] ft. |
| 4. Top width: | 14 ft. |
| 5. Side slopes: | 2H to 1V Upstream
3H to 1V Downstream |
| 6. Zoning: | N/A |
| 7. Impervious Core: | N/A |
| 8. Cutoff: | N/A |
| 9. Grout curtain: | N/A |
| 10. Other: | 3 foot thick and
200 foot wide impervious
upstream blanket |

h. Diversion and Regulating Tunnel N/A

i. Spillway

- | | |
|------------------------|--|
| 1. Type: | Concrete ogee weir
and rectangular
chute with dissipator
and stilling basin |
| 2. Length of weir: | 80 ft. |
| 3. Crest el.: | 1140 |
| 4. Gates: | N/A |
| 5. Upstream Channel: | 86 foot wide approach
channel |
| 6. Downstream Channel: | 70 ft. wide trapezoidal
dumped rock channel
to streambed |
| 7. General: | 4'x5' sluice gate for
fishway on left side
of spillway |

- j. Regulating Outlets - Outlets are the mid-depth and low level pipes connecting at the valve chamber with the 24 inch ductile iron outlet pipe. The mid-depth low flow system has a capacity of 2.5 cfs and the 24 inch outlet pipe has an estimated capacity of 70 cfs.

- | | |
|-----------------------|--|
| 1. Invert: | low-depth outlet - 1116 ⁺
mid-depth outlet - 1128 ⁺ |
| 2. Size: | low-depth outlet - 24"
mid-depth outlet - 6" |
| 3. Description: | low-depth outlet - ductile
iron pipe
mid-depth outlet - steel pip |
| 4. Control Mechanism: | Hand operated valves
on 24" and 6" pipes |
| 5. Other: | hand operated floor
stand type sluice
gate for fishway |

SECTION 2: ENGINEERING DATA

2.1 DESIGN

a. Available Data - The available data consists of original drawings, correspondence, calculations and specifications by Anderson-Nichols and Company, Inc. Drawings and calculations showing changes to original design were available from E. D'Appolonia Consulting Engineers, Inc. Also, there was correspondence concerning inspections and design from the State of Connecticut Water Resources Commission, West Goshen Realty Association, Inc. and Boise Cascade Properties, Inc.

b. Design Features - The drawings, correspondence, calculations and specifications indicate the design features stated in Section 1.

c. Design Data - Design data consists of design calculations, boring logs and drawings by Anderson-Nichols and E. D'Appolonia as listed in "Existing Plans" or "Data and Correspondence" in Appendix B.

2.2 CONSTRUCTION

a. Available Data - Information as contained in any plans, drawings, or specifications previously listed in "Design Data" or Appendix B.

b. Construction Considerations - The dam itself was built as designed except for the 6 inch low flow augmentation system and an addition of a platform and ladder cage in the valve chamber. A diversion dike was also constructed upstream to facilitate construction..

2.3 OPERATIONS

Lake level readings are not taken on any regular schedule. It is reported that the dam spillway capacity has never been exceeded, and no formal operations procedures are known to exist.

2.4 EVALUATION

a. Availability - Existing data was provided by the State of Connecticut Department of Environmental Protection, the Owner and E. D'Appolonia Consulting Engineers, Inc. The Owner made the operations available for visual inspection.

b. Adequacy - Detailed hydrologic/hydraulic data was available and was used to perform computations of spillway capacity. The detailed engineering data required to perform an in-depth stability analysis of the dam was not available. The final assessment of the dam, therefore, must be based primarily on visual inspection, performance history, and spillway capacity computations.

c. Validity - A comparison of records, data, and visual observations reveals no observable significant discrepancies in the record data.

SECTION 3: VISUAL INSPECTION

3.1 FINDINGS

a. General - The general condition of the dam is good. The inspection did reveal some areas requiring attention. The reservoir level was at elevation 1140.5+, 7.5 feet below the top of the dam. There was flow over the weir during the inspection so the spillway could not be observed completely. The weather was cool, wet and cloudy.

b. Dam

Crest - The 14 foot wide crest of the main embankment is gravel and grass covered (Photo 1). The 180+ foot long left embankment (left side of the dam from the spillway) has a grass cover only (Photo 2). No cracks or misalignment of the crest was observed. Several young trees are located on the top of the left embankment.

Upstream Slope - The upstream slope inclination is 2 horizontal to 1 vertical, and protection of the slope is 18 inch thick dumped rock riprap placed between the dam crest and elevation 1135. The riprap is generally in very good condition (Photo 1). The upstream slope of the left embankment has a grass cover (Photo 2) on which an erosion spot of 15+ feet by 12+ feet and 1 to 2 feet in depth was found. The origin of the erosion is probably wave action.

Downstream Slope - The slope inclination is 3 horizontal to 1 vertical and the slope protection is 6 to 10 inch thick seeded topsoil. (Photo 9) There is a 4 foot wide stone surface drain which runs approximately 20 feet from and parallel to the longitudinal drain.

The downstream slope is in good condition. No cracks, sloughing or signs of seepage were detected. The stone drain however, is overgrown with grass and requires maintenance (Photo 3).

Adjacent to the toe of the dam is an extensive swamp area (Photo 3). The origin of this area is not clear although it seems the main water contribution is storm runoff and groundwater from hilly territory surrounding the toe of the dam (Photo 4).

Before completion of the dam construction, five open-system piezometers were installed for monitoring seepage into the foundation. Four piezometers (number 1,2,3,5) were located at the central part of the embankment and one (number 4) was located in the area of the drainage well (Sheet B-1). Number five piezometer, located at the upstream toe, was destroyed during the ice break-up in spring 1970. Five piezometer readings were taken during filling of the reservoir from February, 1970 to January, 1971. A check of the piezometers and some readings and soundings were made during the inspection. The inspection showed that only one piezometer (number 1) has a pipe cap. Two piezometers (numbers 3 and 4) have damage to the external steel pipes (photo 5), which prohibited measurement of their depths.

Piezometer Number	#1		#2		#3		#4	
Reading Date	1/71	5/79	1/71	5/79	1/71	5/79	6/70	5/79
Elevations:								
Top of pipe	1151	1151	1132.7	1132.7	1123.7	1123.7	1121.0	1121.0
Bottom of pipe	1106.0	1117.9	1110.0	1109.1	1100.0	--	1105.5	--
Piezometric surface	1127.0	1127.3	1119.5	1118.1	1120.3	1118.1	1119.1	1117.1
Reservoir level	1138.9	1140.5	1138.9	1140.5	1138.9	1140.5	1138.4	1140.5

Note: Bent pipes at #3 and #4 did not allow measurement of their depths. For original readings see page B-58.

The total seepage discharge from the drainage well outlet, measured in a 90 degree vee-notch weir installed on the manhole cover, is 28.5+ gallons per minute (gpm). Previous measurements of the flow were 22.4+ gpm (in March 1971) and 35+ gpm (in June 1972). These data of seepage indicate no substantial increase in the discharges and hence, no increase in the permeability of the dam.

Spillway - The spillway is the 80 foot long and 5 foot high uncontrolled concrete ogee weir. Water from the spillway weir is conveyed by a 40 foot wide concrete rectangular chute with a stilling basin and a dumped rock channel extending to the existing channel of the Marshepaug river. At the left side of the spillway there is a fishway with a chamber and sluice gate.

The spillway is generally in good condition (Photo 6). The concrete shows no substantial deterioration, misalignment of the construction joints, or seepage spots. Cracks were discovered in the construction joints and corners of the training walls, and range in size from 1/32 to 5/32 inches.

c. Appurtenant Structures - The concrete valve chamber (Photo 7), the concrete low level outlet headwall, and the low level diversion channel (photo 8) are in good condition. No cracking or spalling of the concrete structures, or obstructions in the channel were observed.

d. Reservoir Area - The area surrounding the reservoir is wooded and largely undeveloped. No visible erosion or deterioration of the banks were noted.

e. Downstream Channel - The downstream channel is the natural streambed of the Marshepaug River. The banks are flat to steep and covered with trees and brush (Photo 10). Several wet areas and slight seepage spots were identified on the left bank approximately 200-300 feet from the end of the spillway channel. These areas are probably caused by storm runoff and groundwater from the surrounding terrain.

3.2 EVALUATION

Based upon the visual inspection, the dam is assessed as being generally in good condition. The following features which could influence the future condition and/or stability of the dam were identified.

1. Erosion of the upstream slope of the left embankment can lead to increasingly extensive seepage through the body of the dam.
2. The swamp area at the right portion of the toe of the embankment, if it expands toward the downstream slope, could affect the stability of the dam.

3. Damaged piezometers impair observation of embankment conditions and behavior in the future.
4. Cracks in the concrete spillway training walls could lead to extensive deterioration, thereby compromising the stability of the walls.
5. Seepage data and piezometer readings as listed on page 11 indicate no substantial seepage increases, therefore no increase in the permeability of the dam.

SECTION 4: OPERATIONAL PROCEDURES

4.1 REGULATING PROCEDURES

Lake level readings are not taken on a regular basis. The lake level is dropped 5 feet every winter to allow maintenance to lake front property. The 6 inch low-flow system, which has a design capacity of 2.5 cfs, is used to augment flow downstream during excessively dry summer months.

4.2 MAINTENANCE OF DAM

The dam is kept clear of brush, and the grass is cut several times a year. There is no formal inspection program in existence.

4.3 MAINTENANCE OF OPERATING FACILITIES

Maintenance consists of the operation of the valves when lowering or raising the lake level and for augmentation of downstream flows and greasing the valves periodically. No formal program is known to exist.

4.4 DESCRIPTION OF ANY FORMAL WARNING SYSTEM IN EFFECT

No formal warning system is in effect.

4.5 EVALUATION

The operation and maintenance procedures are generally good, however a formal program of operation and maintenance procedures should be implemented, including documentation to provide complete records for future reference. Also, a formal warning system should be developed and implemented within the time frame indicated in Section 7.1c. Remedial operation and maintenance recommendations are presented in Section 7.

SECTION 5: HYDRAULIC/HYDROLOGIC

5.1 EVALUATION OF FEATURES

a. General - The project is basically a low surcharge storage - high spillage earth embankment, constructed to impound water for recreational use only. The spillway is fairly large and will pass 81% of the project test flood with the dam overtopped by 0.3 feet. A small dam constructed upstream at Tyler Lake will affect inflow to Woodridge Lake as indicated in Appendix D.

b. Design Data - Design data available was the report by Anderson-Nichols and Company, Inc. titled "Hydraulic and Hydrologic criteria for Design of Seven Farms Lake Dam" dated May 18, 1966. See Appendix B, "Engineering Data and Correspondence." Computations, data and graphs for flood routing of Tyler Lake and Woodridge Lake are presented.

c. Experience data - No information on serious problem situations arising at the dam were found, and it does not appear the dam has been overtopped.

d. Visual Observations - No obstructions in the spillway channel or outlets were observed.

e. Test Flood Analysis - The test flood for this high hazard, intermediate size dam is equivalent to the Probable Maximum Flood (PMF). Based upon "Preliminary guidance for Estimating Maximum Probable Discharge", dated March, 1978, peak inflow to the reservoir is 12,600 cfs (Appendix D-1); peak outflow is 7850 cfs with the dam overtopped .3 feet (Appendix D-4). Based upon our hydraulics computations, the spillway capacity is 6340 cfs, which is approximately 81% of the routed Test Flood outflow at the top of the dam.

f. Dam Failure Analysis - Utilizing the April, 1978, "Rule of Thumb Guidance for Estimating Downstream Dam Failure Hydrographs", the peak failure outflow from the dam breaching would be 60,000 cfs. A breach of the dam would result in a rise of 12 feet in the water level of the stream at the initial impact area, which is one quarter of a mile downstream from the dam. This 12 foot rise in flood stage corresponds to an increase in flow of 60,000 cfs and an increase in the water level from a depth of 8.5 feet just before the breach, to a depth of 20.5 feet just after the breach. The rapid 12 foot increase in the water level at the initial impact area would inundate 7 houses to a depth of 8+ feet. Houses along the Marshepaug River approximately 2 miles downstream from the dam at the town of Milton could also be subject to flooding should a breach of the dam occur.

SECTION 6: STRUCTURAL STABILITY

6.1 EVALUATION OF STRUCTURAL STABILITY

a. Visual Observations - The visual inspection did not reveal any indications of stability problems. There are some areas of seepage in the dam embankment and minor cracking in the spillway, as described in Section 3, however they are not considered stability concerns.

b. Design and Construction Data - A 3 foot thick impervious blanket was installed at the upstream toe of the dam and a drainage blanket with pressure relief wells was installed on the downstream slope and toe of the dam. Five piezometers (see Page B58) and a low flow augmentation system were installed during construction of the dam. A continuous program of inspection was also instituted during the construction of the dam (See Appendix B).

c. Operating Records - The operating records available do not include any indication of dam instability since its construction in 1970.

d. Post Construction Changes - There are no records available concerning any post-construction changes of the dam.

e. Seismic Stability - The dam is in Seismic Zone 1 and according to the Recommended Guidelines, need not be evaluated for seismic stability.

SECTION 7: ASSESSMENT, RECOMMENDATIONS AND REMEDIAL MEASURES

7.1 DAM ASSESSMENT

a. Condition - Based upon the visual inspection of the site and past performance, the dam appears to be in good condition. No evidence of structural instability was observed in the dam or its appurtenances. The embankment is generally in good condition with areas of minor concern, such as maintenance and monitoring problems.

Based upon "Preliminary Guidance for Estimating Maximum Probable Discharge" dated March, 1978, peak inflow to the reservoir is 12600 cfs; peak outflow is 7850 cfs with the dam overtopped by .3 feet. Based upon our hydraulics computations, the spillway capacity is 6340 cfs, which is equivalent to approximately 81% of the routed Test Flood outflow.

b. Adequacy of Information - The information available is such that an assessment of the condition and stability of the dam must be based solely on visual inspection, past performance of the dam, and sound engineering judgement.

c. Urgency - It is recommended that the measures presented in Section 7.2 and 7.3 be implemented within two years of the owner's receipt of this report.

d. Need for Additional Information - There is a need for more information as recommended in Section 7.2

7.2 RECOMMENDATIONS

It is recommended that further studies be made by a registered professional engineer qualified in dam design and inspection pertaining to the following:

1. Inspection of the dam during times of low head, as well as high head, to check observable seepage and the condition of the spillway. An evaluation of the significance of the seepage, as well as the condition of the spillway should be undertaken, and any necessary recommendations made by the engineer and implemented by the owner. The engineer should also check piezometers for any damages, to insure proper operating conditions.
2. The swamp area on the right side of the toe of the dam and all streams flowing to this area should be delineated and inspected periodically.

7.3 REMEDIAL MEASURES

a. Operation and Maintenance Procedures - The following measures should be undertaken within the time frame indicated in Section 7.1.c, and continued on a regular basis.

1. Round-the-clock surveillance should be provided by the owner during periods of unusually heavy precipitation or high project discharge. The owner should develop a downstream warning system in case of emergencies at the dam.
2. A formal program of operation and maintenance procedures should be instituted and fully documented to provide accurate records for future reference.
3. A program of inspection by a registered, professional engineer qualified in dam inspection should be instituted on an annual basis. The inspections should be comprehensive in nature and should include the operation of the low level outlet works.
4. Damaged piezometers should be restored and elevations to the top of the pipes should be checked and recorded. The phreatic surface in the foundation of the dam and seepage from the toe drainage system should be monitored periodically by existing piezometers and metering weir. Any substantial change in flow or piezometric levels should be evaluated immediately.
5. The cutting of grass on the downstream slope and the toe of the dam should be continued as part of the routine dam maintenance. Trees on the crest of the dam and any vegetation on the stone surface drain at the dam toe should be removed.
6. The eroded area on the upstream face of the left embankment should be repaired, and riprap placed to eliminate further erosion.
7. Cracks on concrete surfaces of the spillway training walls should be repaired.

7.4 ALTERNATIVES

This study has identified no practical alternative to the above recommendations.

APPENDIX A

INSPECTION CHECKLIST

VISUAL INSPECTION CHECK LIST

PARTY ORGANIZATION

PROJECT Woodridge Lake Dam

DATE: May 3, 1979

TIME: 2:00 p.m.

WEATHER: Cloudy, 65°F

W.S. ELEV. 1140.5 U.S. DN.S

PARTY:

INITIALS:

DISCIPLINE:

1. <u>Peter M. Heynen</u>	<u>PMH</u>	<u>Cahn Engineers, Inc.</u>
2. <u>Calvin R. Goldsmith</u>	<u>CRG</u>	<u>Cahn Engineers, Inc.</u>
3. <u>Miron Petrovsky</u>	<u>MP</u>	<u>Cahn Engineers, Inc.</u>
4. <u>George Stephens</u>	<u>GS</u>	<u>Cahn Engineers, Inc.</u>
5. <u>Jay Costello</u>	<u>JC</u>	<u>Cahn Engineers, Inc.</u>
6. <u>Tulli Amicone</u>	(Owner Representative) <u>Woodridge Lake Property</u> <u>Owner's Assoc.</u>	

PROJECT FEATURE

INSPECTED BY

REMARKS

1. <u>Earth Embankment</u>	<u>PMH, CRG, MP, GS, JC</u>	
2. <u>Earth Dike</u>	<u>PMH, MP</u>	
3. <u>Spillway and Channel</u>	<u>PMH, MP, GS, JC</u>	
4. <u>Upper Gate Chamber</u>	<u>PMH, GS, JC</u>	
5. <u>Low Level Outlet & Channel</u>	<u>PMH, MP, GS, JC</u>	
6. _____		
7. _____		
8. _____		
9. _____		
10. _____		
11. _____		
12. _____		

PERIODIC INSPECTION CHECK LIST

Page A-2PROJECT WOODRIDGE LAKE DAMDATE May 3, 1979PROJECT FEATURE EARTH Main Dam Embankment BY PMH, CRG, MP, G.S, JC

AREA EVALUATED	CONDITION
<u>DAM EMBANKMENT</u>	
Crest Elevation	1148.0
Current Pool Elevation	1140.5 ±
Maximum Impoundment to Date	N/A
Surface Cracks	NONE OBSERVED
Pavement Condition	Good, graveled cover
Movement or Settlement of Crest	NONE OBSERVED
Lateral Movement	NONE OBSERVED
Vertical Alignment	NONE OBSERVED
Horizontal Alignment	NONE OBSERVED
Condition at Abutment and at Concrete Structures	Good
Indications of Movement of Structural Items on Slopes	N/A
Trespassing on Slopes	NONE
Sloughing or Erosion of Slopes or Abutments	NONE OBSERVED
Rock Slope Protection-Riprap Failures	NONE OBSERVED
Unusual Movement or Cracking at or Near Toes	NONE OBSERVED
Unusual Embankment or Downstream Seepage	SNAMP area at RIGHT SIDE OF DAM TOE
Piping or Boils	NONE OBSERVED
Foundation Drainage Features	Drainage trench with toe drain and relief wells
Toe Drains	
Instrumentation System	Piezometers and metering weir

A-2

PERIODIC INSPECTION CHECK LIST

Page A-3PROJECT Woodridge Lake DamDATE May 3, 1979PROJECT FEATURE Earth dikeBY PMH, MP

AREA EVALUATED	CONDITION
<u>DIKE EMBANKMENT</u>	
Crest Elevation	1148.0
Current Pool Elevation	1140.5
Maximum Impoundment to Date	N/A
Surface Cracks	None observed
Pavement Condition	Good, grassed cover
Movement or Settlement of Crest	None observed
Lateral Movement	None
Vertical Alignment	None
Horizontal Alignment	None
Condition at Abutment and at Concrete Structures	Good
Indications of Movement of Structural Items on Slopes	None
Sloughing or Erosion of Slopes or Abutments	Substantial erosion area on upstream face
Rock Slope Protection-Riprap Failures	N/A
Unusual Movement or Cracking at or Near Toes	None observed
Unusual Embankment or Downstream Seepage	None observed
Piping or Boils	None observed
Foundation Drainage Features	N/A
Toe Drains	N/A
Instrumentation System	N/A
Trespassing on Slopes	None

PERIODIC INSPECTION CHECK LIST

Page A-4PROJECT Woodridge Lake DamDATE May 3, 1979PROJECT FEATURE Intake channel and structure BY PMHGS, JC

AREA EVALUATED		CONDITION
<u>OUTLET WORKS-INTAKE CHANNEL AND</u> <u>INTAKE STRUCTURE</u> a) <u>Approach Channel</u> Slope Conditions Bottom Conditions Rock Slides or Falls Log Boom Debris Condition of Concrete Lining Drains or Weep Holes b) <u>Intake Structure</u> Condition of Concrete Stop Logs and Slots		
		 <i>Short 12' wide channel</i> <i>Under water</i> <i>Under water</i> <i>Under water</i> <i>N/A</i> <i>Not observed</i> <i>N/A</i> <i>N/A</i> <i>Concrete structure</i> <i>Under water</i> <i>Under water</i>

PERIODIC INSPECTION CHECK LIST

Page A-5PROJECT Woodridge Lake DamDATE May 3, 1979PROJECT FEATURE Concrete valve chamberBY PMH, GS, JC

AREA EVALUATED	CONDITION
<u>OUTLET WORKS-CONTROL TOWER</u>	
a) <u>Concrete and Structural</u>	
General Condition	Good
Condition of Joints	Not observed
Spalling	None observed
Visible Reinforcing	None
Rusting or Staining of Concrete	None
Any Seepage or Efflorescence	None observed
Joint Alignment	Not observed
Unusual Seepage or Leaks in Gate Chamber	None observed
Cracks	None observed
Rusting or Corrosion of Steel	None
b) <u>Mechanical and Electrical</u>	
Air Vents	} N/A
Float Wells	
Crane Hoist	
Elevator	
Hydraulic System	
Service Gates	24" and 6" gate valves, operable conditions
Emergency Gates	} N/A
Lightning Protection System	
Emergency Power System	
Wiring and Lighting System	

PERIODIC INSPECTION CHECK LIST

Page A-6PROJECT Woodridge Lake DamDATE May 3, 1979PROJECT FEATURE Low level outlet and channelBY PMH, MP, GS, JC

AREA EVALUATED	CONDITION
<u>OUTLET WORKS-OUTLET STRUCTURE AND OUTLET CHANNEL</u>	<i>Concrete headwall and riprap faced. channel</i>
General Condition of Concrete	<i>Good</i>
Rust or Staining	<i>None observed</i>
Spalling	<i>None observed</i>
Erosion or Cavitation	<i>None</i>
Visible Reinforcing	<i>None</i>
Any Seepage or Efflorescence	<i>None observed</i>
Condition at Joints	<i>Not observed</i>
Drain Holes	<i>N/A</i>
Channel	<i>12' wide stone trench</i>
Loose Rock or Trees Overhanging Channel	<i>None observed</i>
Condition of Discharge Channel	<i>Good</i>

PERIODIC INSPECTION CHECK LIST

Page A-7

PROJECT Woodridge Lake Dam

DATE May 3, 1979

PROJECT FEATURE Spillway and channels

BY PMH, MP, GS, JC

AREA EVALUATED	CONDITION
<u>OUTLET WORKS-SPILLWAY WEIR, APPROACH AND DISCHARGE CHANNELS</u>	
a) <u>Approach Channel</u>	86' dumped rock channel
General Condition	Good
Loose Rock Overhanging Channel	None
Trees Overhanging Channel	None
Floor of Approach Channel	Under water
b) <u>Weir and Training Walls</u>	Concrete ogee weir and chute
General Condition of Concrete	Good
Rust or Staining	None observed
Spalling	Cracks of construction joints
Any Visible Reinforcing	None
Any Seepage or Efflorescence	None observed
Drain Holes	Under water
c) <u>Discharge Channel</u>	To' dumped rock and earth channel
General Condition	Good
Loose Rock Overhanging Channel	None
Trees Overhanging Channel	None
Floor of Channel	Good
Other Obstructions	None observed

APPENDIX B

ENGINEERING DATA AND CORRESPONDENCE

WOODRIDGE LAKE DAM

EXISTING PLANS

"Topography and Dam Sites"
Malloy, Davis and Storch
West Hartford, Conn.

"Layout of Dam"
Anderson-Nichols and Co., Inc.
Boston, Mass.
May 1966

"Seven Farms Lake Dam"
Anderson-Nichols and Co., Inc.
Boston, Mass.
July, 1966 (set of 10)

"Seven Farms Lake Dam"
Ogee Spillway Design and Stability Computations
Exhibits A,B,C,D,E,
Anderson-Nichols and Co., Inc.
Boston, Mass. (1966)

"Construction Specifications"
Anderson-Nichols and Co., Inc.
Boston, Mass. (1966)

"Seven Farms Lake"
E. D'Appolonia Consulting Engineers, Inc.
Pittsburgh, Pa.
July, 1969 (set of 4)

"Earth Dam and Spillway Construction"
E. D'Appolonia Consulting Engineers, Inc.
Pittsburgh, Pa. (July 1969)

"Diversion Calculations"
E. D'Appolonia Consulting Engineers, Inc.
Pittsburgh, Pa. (Sept. 1969)

SUMMARY OF DATA AND CORRESPONDENCE

<u>DATE</u>	<u>TO</u>	<u>FROM</u>	<u>SUBJECT</u>	<u>PAGE</u>
July 30, 1963	Leroy Simmons Goshen First Select- men Office	P.C. Hyzer - Brigadier General, Division Engineer Army Corps of Engineers	Federal grant or assistance for dam construction	B-4
Nov. 20, 1964	Richard H. Meritt	Anderson-Nichols and Co. Inc.	Preliminary design figures and cost estimates	B-6
May 18, 1966	John J. Curry	Anderson-Nichols and Co., Inc.	Design Specifications and hydrologic consideration	B-8
July 19, 1966	Water Resources Commission	West Goshen Realty Assoc., Inc.	Application for construc- tion	B-27
Sept. 20 1966	West Goshen Realty Assoc., Inc.	Water Resources Commission	Construction permit	B-29
May 27, 1969	Water Resources Commission	E. D'Appolonia Consulting Engineers, Inc.	Proposed changes to design and negotiations for purchase of dam and land by Boise Cascade Properties, Inc.	B-30
Sept. 23, 1969	H. Robert Hoffman	Richard D. Ellison	Diversion Dike	B-33
Nov. 18, 1969	H. Robert Hoffman	Richard D. Ellison	Design calculations for a low-flow augmentation system	B-35
March 31, 1970	File	William H. O'Brian III Water Resources Commission	Semi-final inspection report	B-56

<u>DATE</u>	<u>TO</u>	<u>FROM</u>	<u>SUBJECT</u>	<u>PAGE</u>
March 24, 1971	William H. O'Brian III	E. D'Appolonia Consulting Engineers, Inc.	Pizometer readings during filling of reservoir	B-57
Sept. 15, 1971	Michael J. Taylor	William H. O'Brian III	Ladder cage and platform details	B-59
July 11, 1972	Dept. of Environ- mental Protection	Macchi and Hoffman, Engrs.	Final inspection report	B-62
Dec. 13, 1972	Boise Cascade Properties, Inc.	Water Resources Commission	Certificate of approval	B-64

*I was able to obtain this before mailing
letter to you so am sending it all together.*

NEDGE

Dick Merrill

30 July 1963

Mr. LeRoy Simons
Office of Selectmen
Town of Goshen
Goshen, Connecticut

Dear Mr. Simons:

Please refer to your letter of 8 May 1963 in which the Town requested Federal assistance in studies for a possible multi-purpose flood control and recreation dam on the Marshpaug River in the vicinity of Tyler Pond at Goshen, Connecticut. We have also received your letter of 18 July 1963 which requests a three-month postponement of our studies.

You will recall that on 28 June 1963 members of my staff met with you and Mr. Richard C. Kobylanski, Second Selectman for the Town of Goshen, to discuss the above matter. Our studies which have been in progress since that meeting are now completed.

Existing authorities permit Federal participation in projects with flood control and recreation provided that benefits attributable to recreation do not exceed 50 percent of the annual project costs. Our findings at Goshen conclude project benefits would come almost entirely from recreation and consequently, Federal participation cannot be recommended at this time.

On the basis of the data supplied by the town the multi-purpose flood control and recreation dam which was studied, would have a permanent recreation pool at elevation 1145 mean sea level (m.s.l.) and a flood control spillway at elevation 1151 m.s.l. The recreation pool would have a water surface area of 427 acres and a maximum depth of about 25 feet. The dam which would provide flood control for a drainage area of about nine square miles, would have a top elevation of 1160 m.s.l. and be about 1000 feet in length, including a concrete spillway of 120 feet. The dam would have a maximum height of 35 feet. The project would require the relocation of approximately 5500 feet of Marshpaug Road. It is estimated that the total project cost would approximate \$300,000.

MEMO

Mr. LeRoy Simons

30 July 1963

Investigation revealed that flooding of properties downstream of the proposed dam site, located primarily in the Village of Milton, is infrequent and during past floods has been minor in nature. Inspection of these properties revealed five bridges, five homes, one church, a parish house and a two-car garage could experience minor flooding in the event of a major or record flood. In view of the sparse development which exists downstream of the proposed dam site, flood control benefits are not of sufficient magnitude to permit Federal participation in accordance with existing regulations.

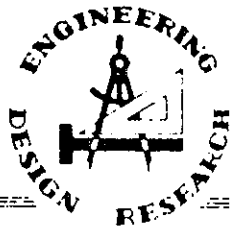
In view of the interest that has been expressed in previous communications with Judge Henry J. Orlando on this matter, I am sending him a copy of this report.

Sincerely yours,

P. C. NYZER
Brigadier General, USA
Division Engineer

cc: **Judge Henry J. Orlando**
111 Franklin Square
New Britain, Connecticut

Water Resources Commission
State of Connecticut
State Office Building
Hartford, Connecticut



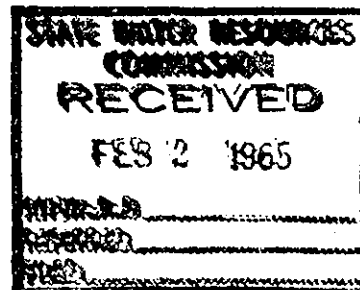
ANDERSON-NICHOLS *Company, Inc.*

A CO-ORDINATED ENGINEERING SERVICE

BOSTON
150 CAUSEWAY STREET
BOSTON 14, MASS.

November 20, 1964

Mr. Richard H. Merritt
Old Farms
Farmington, Connecticut



SUBJECT: Dam and Reservoir at West Goshen
Our Job T-497

Dear Richard:

Thanks for your letter of November 17, 1964 with information as to the required specifications for the relocated road if we use the lower dam site.

We have made some study of your project this week.

We find that the drainage area above the proposed lower dam site is 9.1 square miles and that the area of a pond created by a dam at the lower site, designed to carry the water to elevation 1140, mean sea level, would be 395 acres. The preliminary study indicates that elevation 1140 is the optimum elevation for the pond. Any higher elevation would increase the cost of the dam materially, and would not shorten the length of docks required to reach a usable depth of water for boating to any great extent.

We have made a preliminary estimate of the cost to construct a dam and the required highway at the lower site, and also made an estimate of the cost to construct a dam at the upper site, which we viewed on Tuesday, November 10, 1964.

The dam, in each case, would be designed with a concrete spillway with a crest at elevation 1140 and the top of the earth embankment section at elevation 1148. The spillway would be designed to pass a flood of 5000 cubic feet per second with a pond elevation of 1144. The culvert in the road associated with the lower dam site would also be designed to carry 4000 cubic feet per second.

In each case, the cost of the clearing for the construction required is included. The cost of clearing the reservoir area between the upper and lower sites is not included as the added shore area would be available for sale.

It has been assumed that sufficient impervious material for the core of the earth section of either dam could be found within a short hauling station.

ANDERSON-NICHOLS & COMPANY, Inc.

Mr. Richard H. Merritt
Page Two
November 20, 1964

The estimated cost for the construction of the dam with concrete spillway at the lower site and about 3500 linear feet of relocated highway is \$270,000.

The estimated cost for the construction of the dam with the concrete spillway and a bridge to carry the road over the spillway outlet at the upper site is \$293,000.

The estimates are preliminary and subject to change when more detailed surveys of the area and soil boring samples to determine the foundation conditions are available.

It is our feeling that the upper site would probably require more costly treatment to prevent seepage under the dam than the lower site.

Very truly yours,

ANDERSON-NICHOLS & COMPANY, INC.



Harry M. Nelson

HMN/mlc



ANDERSON-NICHOLS *Company, Inc.*

A COMPREHENSIVE ENGINEERING AND MANAGEMENT SERVICE

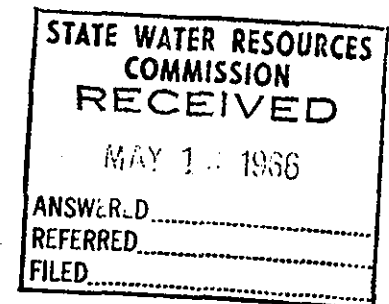
BRANCH OFFICE
HARTFORD, CONNECTICUT

150 CAUSEWAY STREET
BOSTON, MASSACHUSETTS 02114
AREA CODE 617 742-3400

May 18, 1966

Mr. John J. Curry
Chief Engineer
Connecticut Water Resources Commission
State Office Building
Hartford, Connecticut 06115

SUBJECT: Hydraulic and Hydrologic Criteria
for Design of Seven Farms Lake
Goshen, Connecticut
Our Job No. T-497



Dear Mr. Curry:

We have been retained by the West Goshen Realty Association, Inc. of Farmington to design a dam on the Marshepaug River in the Town of Goshen. We are aware that the final construction plans and specifications must meet the requirements of your Agency to obtain a permit for construction.

The hydrologic and hydraulic criteria employed to date has a significant bearing on other design aspects of the structure. Therefore, we are submitting a summary of our recommended hydrologic and hydraulic criteria and the results of the analyses performed to date so as to obtain your comments and approval of this aspect of the design.

General - The dam site is located in the Town of Goshen at approximately 41° 47' 50" Latitude, 73° 15' 00" Longitude and is shown on the Cornwall Quadrangle Map. The watershed is shown on Exhibit I comprising portions of the Cornwall, West Torrington, Norfolk and South Canaan Quadrangle Maps. Of the total 8.89 square miles of drainage area upstream of the dam site, Tyler Lake exerts control of runoff from 6.44 square miles. The topography of the watershed is moderately steep and rural in character with about 40% open and 60% wooded.

Mr. John J. Curry
Connecticut Water Resources Commission
Page Three
May 18, 1966

Design Flood Outflow (Seven Farms Lake) - Storage indication curves were developed for Seven Farms Lake based on Area-Capacity and Stage-Discharge Curves. The latter was predicated on an 80 foot concrete ogee spillway with crest elevation at 1140 feet, m. s. l. and a 4.0 discharge coefficient. The foregoing relationships are shown on Exhibit IV a through d. The total design flood inflow was routed through Seven Farms Lake assuming initial lake level at 1140 feet, m. s. l. to obtain the design flood peak outflow of 3040 cfs (342 csm) and corresponding design high water of 1,144.5 feet, m. s. l. The flood routing is shown on Exhibit IV e and f.

Freeboard - The required freeboard to preclude overtopping of the dam by wind and wave action was computed to be 3.5 feet establishing the elevation of top of dam at 1,148.0 feet, m. s. l. Dumped rock will be placed on the upstream face of the dam to protect against erosive forces of wave action.

Exit Channel and Energy Dissipator - Provisions will be incorporated in the design to convey the design flood from the dam to the natural channel downstream so as to preclude damage to the embankment and to downstream properties. This will be accomplished by a 40 foot wide concrete rectangular channel extending 154 feet to a concrete stilling basin to dissipate the energy. A trapezoidal earth channel approximately 60 feet in length will convey the "stilled" flow to the existing channel of the Marshepaug River.

SUMMARY OF PERTINENT DATA

Location of Dam -

Town of Goshen
Cornwall Quadrangle
41° 47' 50" Latitude - 73° 15' 00" Longitude
Marshepaug River

Name -

Seven Farms Lake

Mr. John J. Curry
Connecticut Water Resources Commission
Page Two
May 18, 1966

Design Concept - The proposed dam will consist of a compacted earth fill embankment having a total length of approximately 1400 feet, a maximum height of 36 feet, top width of 14 feet, side slopes of 1V:2H upstream and 1V:3H downstream. The spillway, consisting of a concrete ogee section, exit channel and energy dissipator will be situated on the southerly portion of the dam. A 36 inch gated outlet will be provided through the dam for purposes of dewatering and low flow releases. The general design concept is shown on Exhibit II.

August 1955 Storm - The rainfall associated with the Hurricane Diane Storm of 1955 on the watershed amounted to approximately 12 inches in 30 hours. Records at Norfolk, representative of the rainfall on the watershed, indicated two distinct periods of significant precipitation comprising 3 inches during 8 hours of the morning on August 18th and 9 inches during 10 hours of the same evening and early morning hours of the 19th. While other portions of the region received greater amounts of rainfall, this storm was apparently the maximum of record for this watershed.

Design Flood - The design flood was developed from design storm rainfall, unit hydrographs and flood routings through Tyler Lake. An initial analysis was made to reproduce the August 1955 flood hydrographs utilizing the experienced rainfall pattern and unit hydrographs for the contributing drainage areas upstream and downstream of Tyler Lake Dam. In recognition of the likelihood of occurrence of an event more severe than experienced in August 1955, a flood twenty-five (25%) percent greater was selected for the design flood.

Tyler Lake Routing - Storage indication curves were developed for Tyler Lake (Exhibit III a through d). The design flood inflow hydrograph was routed through Tyler Lake (Exhibit III e) to obtain the design flood outflow. The results of this routing, shown on Exhibit III f, indicate the significant effect of Tyler Lake storage which reduces the peak inflow from 6480 cfs to 3200 cfs.

Design Flood Inflow (Seven Farms Lake) - The design flood component from the contributing area downstream from Tyler Lake was combined with Tyler Lake Outflow to develop the Design Flood Inflow to Seven Farms Lake. The flood has a peak discharge of 4425 cfs, representing a unit rate of runoff of 498 csm.

Mr. John J. Curry
Connecticut Water Resources Commission
Page Four
May 18, 1966

Owner -

West Goshen Realty Association, Inc.

Dam and Reservoir Use -

Recreation

Drainage Area at Dam -

5688 Acres - 8.89 Square Miles

Upstream Discharge Control -

Tyler Lake - D.A. = 6.44 Square Miles
Pond Surface Area - 189 Acres

Character of Upstream Area -

40% Open Land - 60% Wooded Land
Rural
Moderately Steep Topography

Nature of Bank Area (for 4 Miles Downstream) -

No Appreciable Flood Plain. Relatively Little
Damageable Property.

Lake Area -

At Spillway Crest (El. 1140 feet, m.s.l.) - 390 Acres

Type of Dam -

Earth Embankment

ANDERSON-NICHOLS & COMPANY, INC.

Mr. John J. Curry
Connecticut Water Resources Commission
Page Five
May 18, 1966

Spillway -

Type - Concrete Ogee (uncontrolled)
Crest - Elevation 1140 feet, m.s.l.

Freeboard -

3.5 Feet above Design High Water

Top of Dam -

Elevation 1148 feet, m.s.l.

Maximum Height of Dam -

36 Feet

Length of Dam -

1400 Feet

Dam Foundation -

Earth

Spillway Design Flood -

Basis - 25% Greater than Computed August 1955
Flood of Record
Inflow - 4425 cfs Peak (498 csm)
Outflow - 3040 cfs Peak (342 csm)
Design High Water -
Elevation 1,144.5 feet, m.s.l.
4.5 feet above Spillway Crest

Exit Channel and Energy Dissipator -

Rectangular Concrete Channel and Concrete Stilling Basin

ANDERSON-NICHOLS & COMPANY, INC.

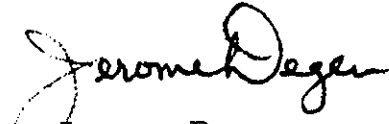
Mr. John J. Curry
Connecticut Water Resources Commission
Page Six
May 18, 1966

The foregoing discussion with accompanying Exhibits constitutes our recommended basis for detailed structural design of the dam. In our opinion, the recommendations presented herein will afford a high degree of protection against the hazard of failure. We would appreciate receiving your comments and concurrence to expedite our completion of final plans and specifications.

Should you desire further information, we will be pleased to furnish it.

Very truly yours,

ANDERSON-NICHOLS & COMPANY, INC.

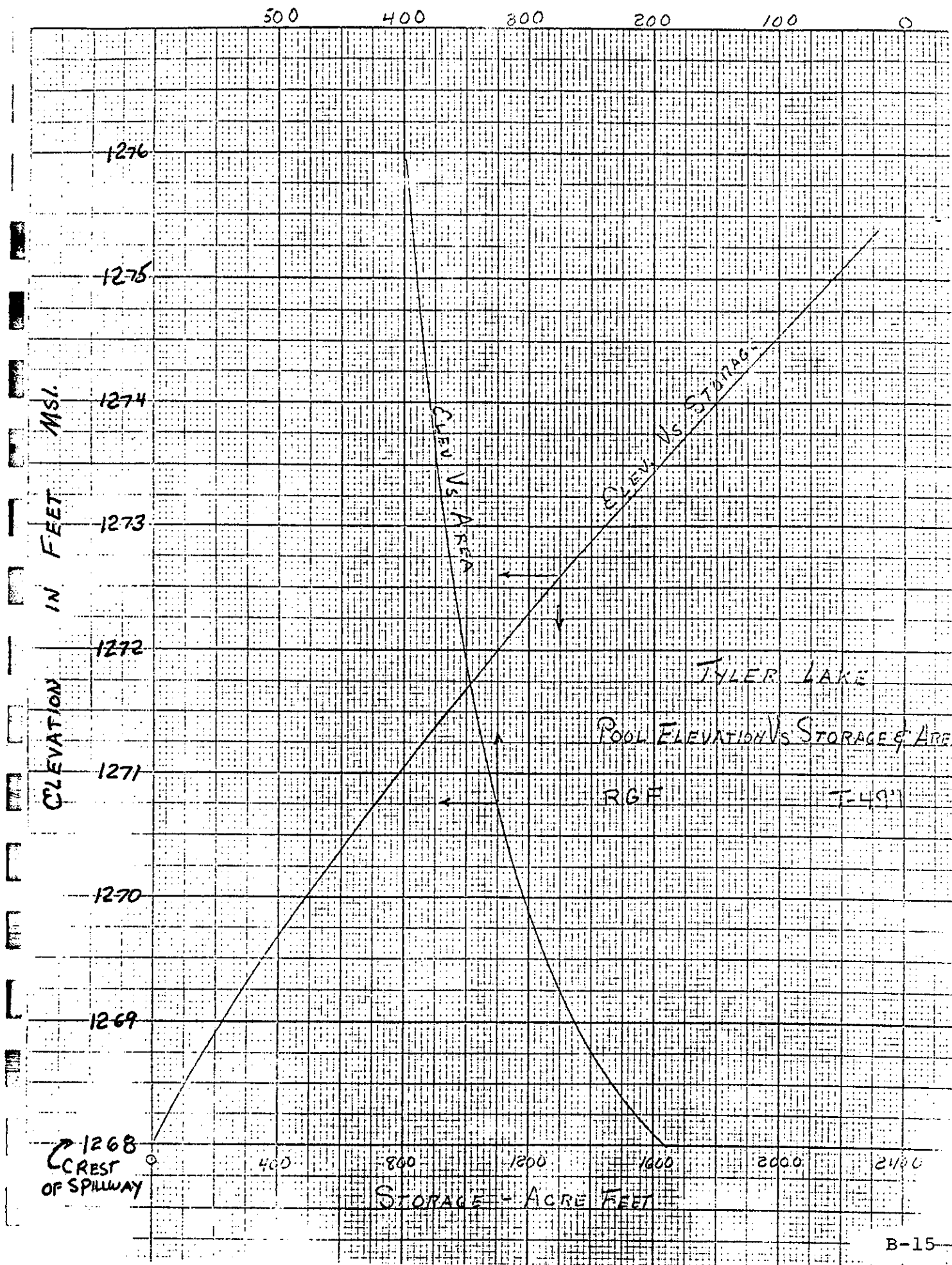

Jerome Degen

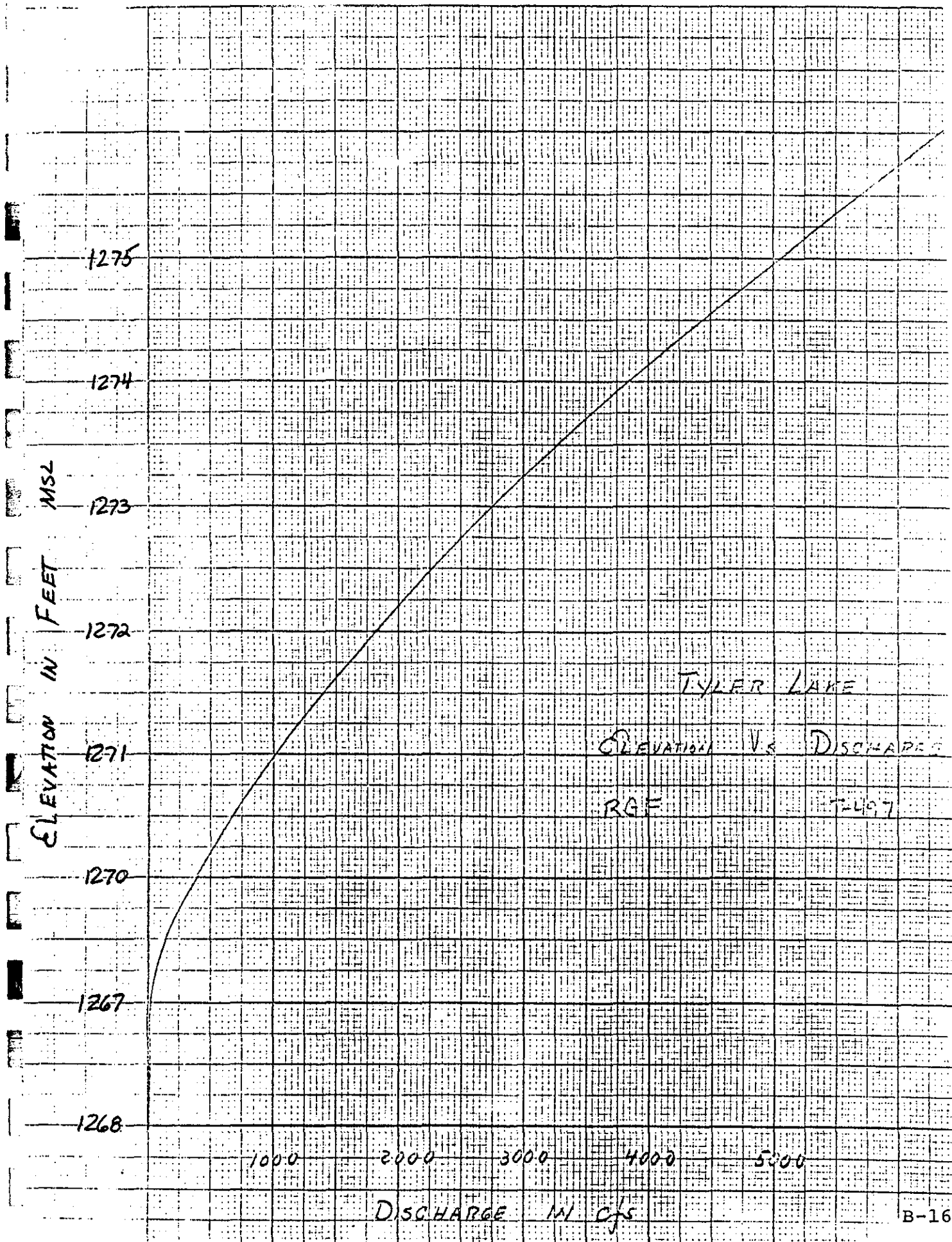
JD/mlc

enc.

ANDERSON-NICHOLS & COMPANY, INC.

B-13





Job No. T-497

TYLER LAKE

SQUARES
IN. SCALE 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26

STORAGE INDICATION

OUTFLOW

ELEV

STORAGE

Acft(x12.1) cfs-hrs

$\Delta T = 1 \text{ hour}$

$\frac{25}{\Delta T} + 0$

0	1268	0	0
5	1268.28	55	666.5
10	1268.5	101	1222
15	.64	134	1621
28	1269	220	2662
50	.1	245	2964
100	.28	292	3533
200	.57	370	4477
400	1270.01	496	6002
800	.7	690	8349
1000	.98	777	9402
1500	1271.62	985	11918
2000	272.2	1165	14096
2500	.74	1355	16396
3000	3.24	1520	18392
3500	.7	1690	20449
4000	4.15	1855	22446
4500	.58	2010	24321
5000	.98	2160	26186
5500	5.37	2310	27951

66902

3600

3200

2800

OUTFLOW - cfs

1600

1200

800

400

0

10

20

30

40

50

60

B-18

25 + 0 - 1000's cfs-hours

EXHIBIT III

TYLER LAKE

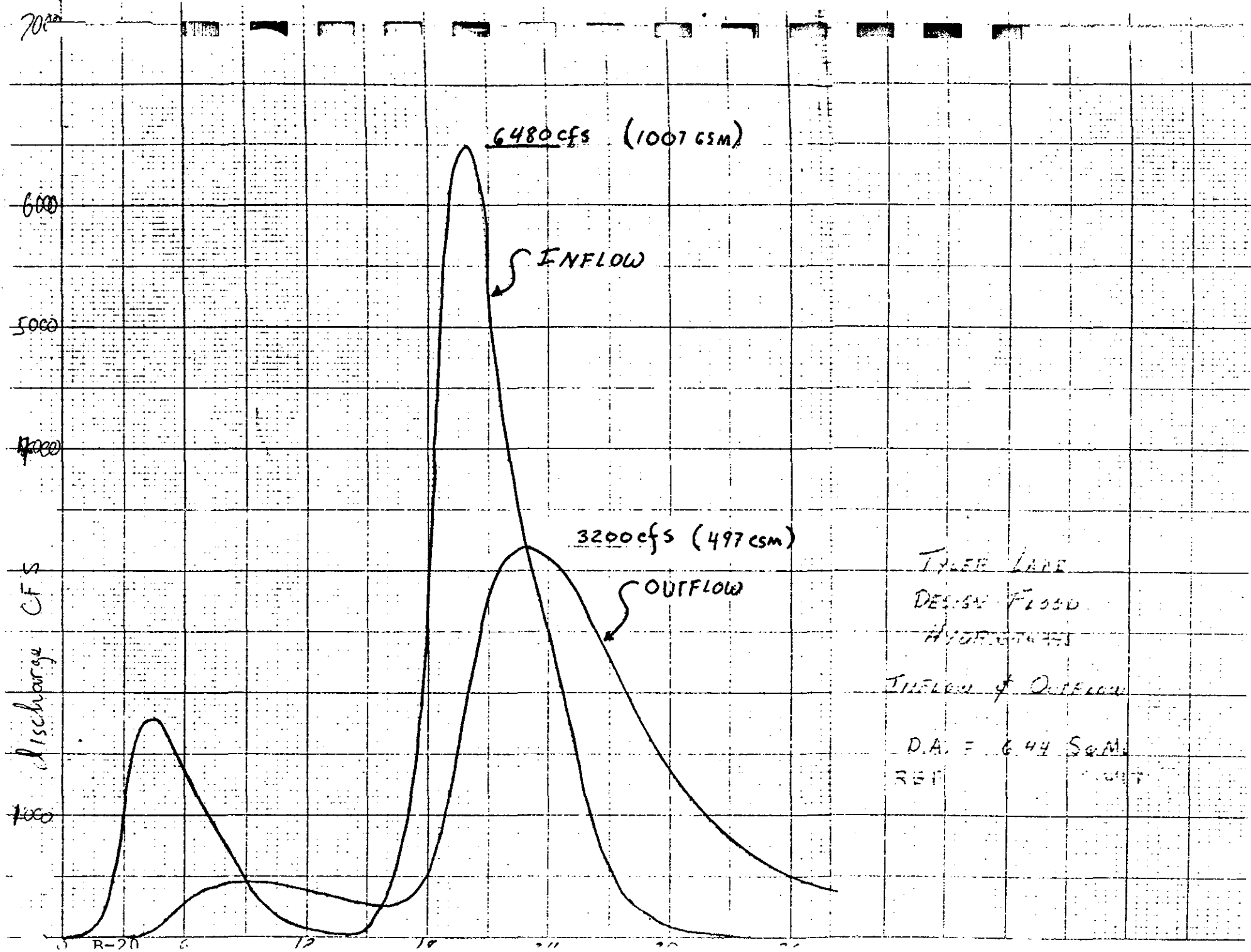
STORAGE-INDICATION CURVE

RGI

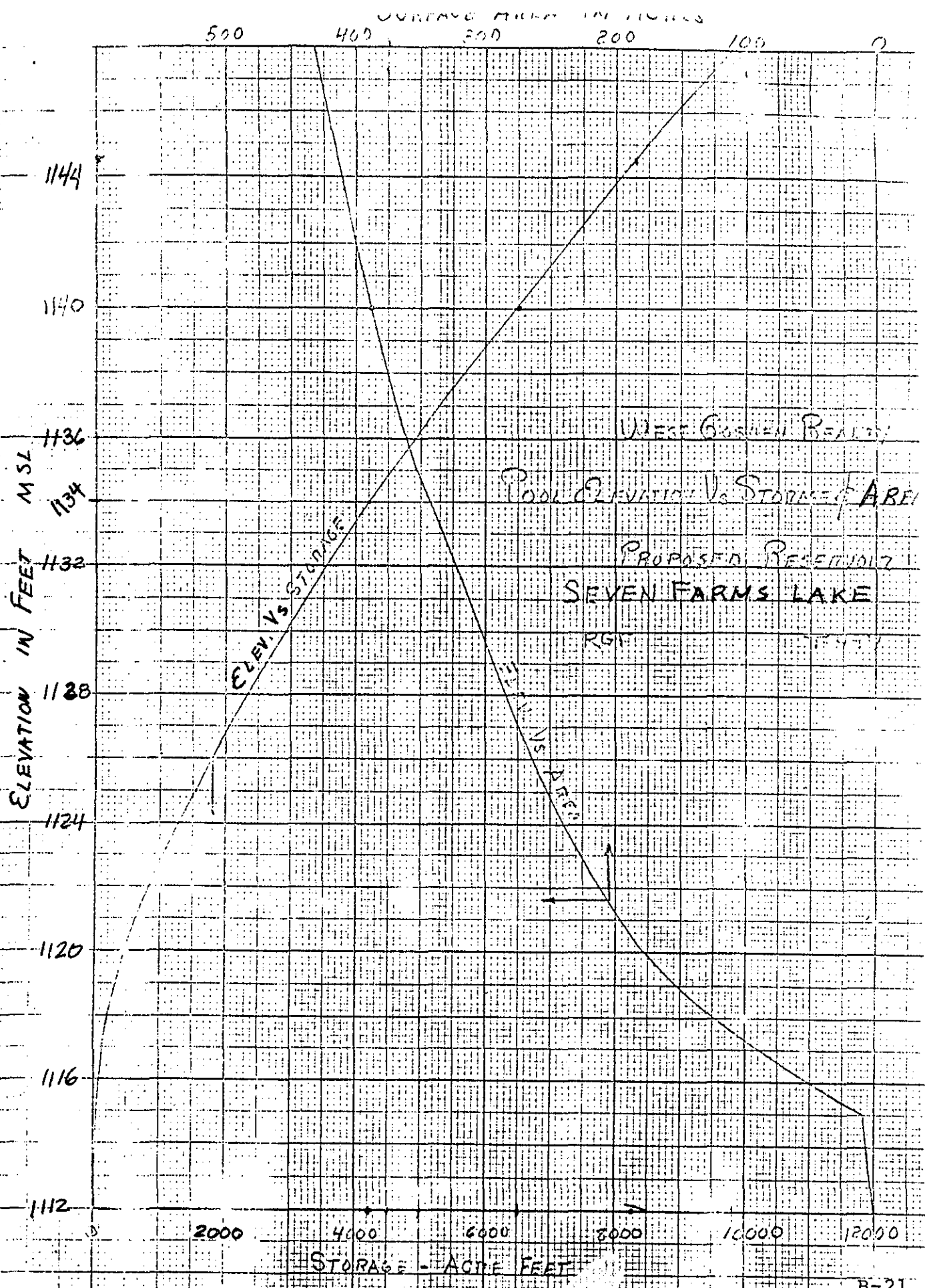
T-49-1

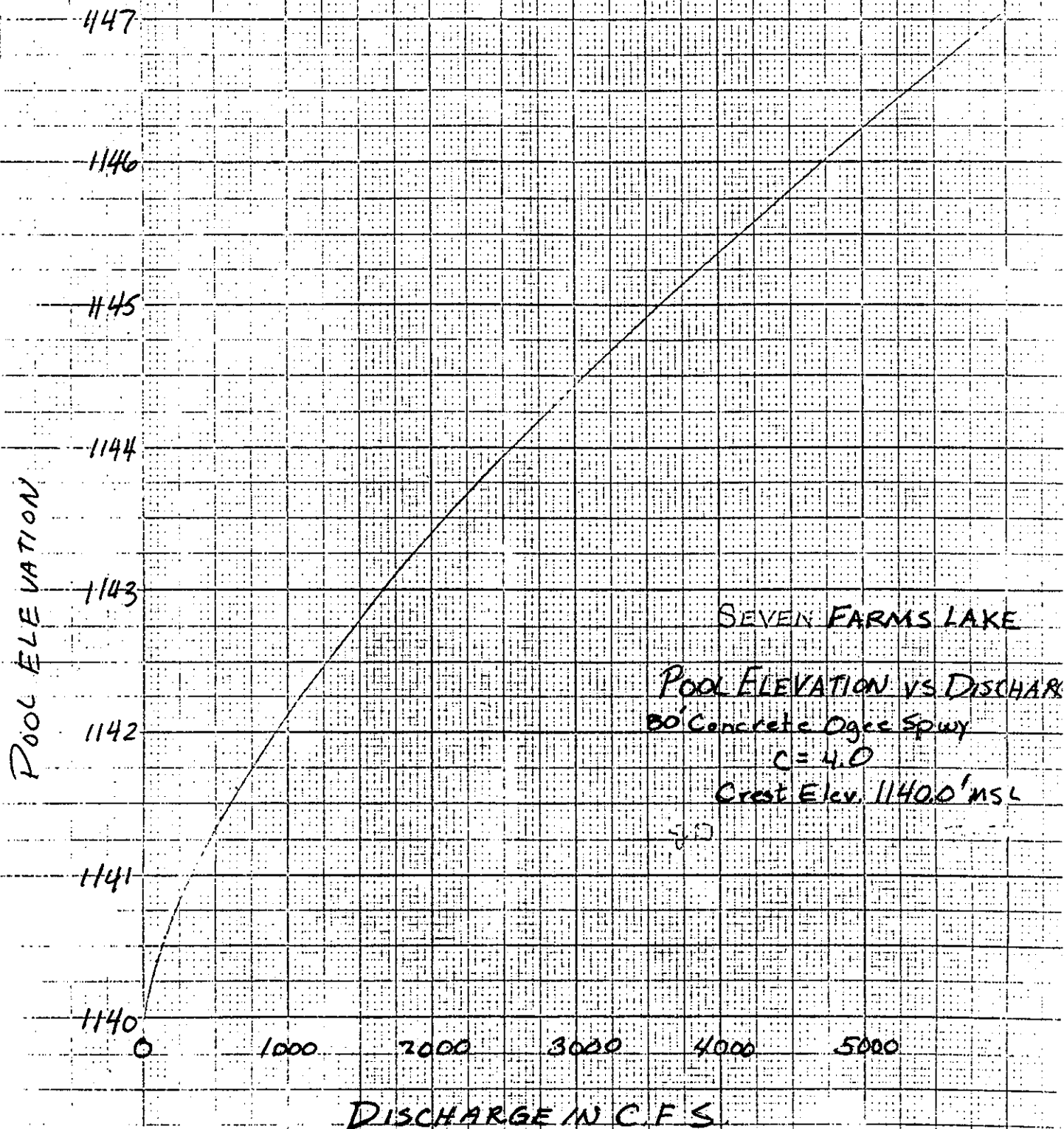
B-19

EXHIBIT D.



TALLE LAKE
DESIGN FLOOD
HYDROGRAPH
INFLOW & OUTFLOW
D.A. = 644 SQ.M.
REF. 1007





Subject WEST DOSHEN REALTY
PROPOSED RESERVOIR
STORAGE-INDICATION

Sheet No. _____
 Date 4 April 1966
 Computed RGF
 Checked JJD

STATION	0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28
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	ELEV.	STORAGE	H	H ^{3/2}	①
		Cfs-hours			= 320 H ^{3/2}
3	1140	0	0	0	0
4	1140.4	1936	.4	.253	51
5	1140.8	3872	.8	.7155	229
6	1141.2	5808	1.2	1.3145	421
7	1141.6	6776	1.6	2.024	645
8	1142.0	9680	2.0	2.828	905
9	1142.4	11616	2.4	3.718	1190
10	1142.8	13552	2.8	4.685	1499
11	1143.2	15488	3.2	5.721	1832
12	1143.6	17424	3.6	6.830	2186
13	1144.0	19360	4.0	8.000	2560
14	1144.5	21788	4.5	9.546	3055
15	1145	24200	5.0	11.18	3576
16	1146		6.0	14.70	4704
17	1147		7.0	18.53	5930
18	1148		8.0	22.63	7242

$$\frac{2s}{\Delta T} + 0$$

$$\Delta T = 1 \text{ hr}$$

OUTFLOW - cfs

3600

3200

2800

2400

2000

1600

1200

800

400

0

10 20 30 40 50 60 B-24

$\frac{25}{1000} + 0 = 1000's \text{ cfs} - \text{hours}$

WEST GOSHAW PEARLY
STORAGE INDICATION CUR
PROPOSED RESERVOIR
SEVEN FARMS AREA
RGT

EXHIBIT IV d

AREA SCALE	0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	
	TIME	OUTFLOW TYLER	LOCAL	INFLOW	$\Delta_1 + \Delta_2$	$\frac{\Delta_1}{2} + 0$	OUTFLOW	EL																							
1	0	0	0	0	0	0	0	114																							
2	1	-	12	12	12	12	-	-																							
3	2	1	175	176	188	200	1	-																							
4	3	7	706	713	829	1027	12	1140																							
5	4	22	844	866	1579	2642	45	1140																							
6	5	126	625	751	1617	4169	87	1140																							
7	6	276	475	751	1502	5497	132	1140																							
8	7	373	356	729	1480	6713	177	1140																							
9	8	430	250	680	1409	7764	221	1140																							
10	9	455	112	567	1247	8569	255	1140																							
11	10	450	38	488	1055	9114	280	114																							
12	11	420	25	445	933	9437	296	1140																							
13	12	380	12	392	837	9732	307	1140																							
14	13	240	6	346	738	9256	313	1140																							
15	14	203	3	206	652	9202	315	1140																							
16	15	277	38	315	621	9273	314	1140																							
17	16	265	219	484	799	10044	322	114																							
18	17	314	700	1014	1498	10898	362	114																							
19	18	510	2406	2916	3930	14104	532	114																							
20	19	1115	2250	4365	7281	20321	915	114																							
21	20	2650	2375	4425	8790	27281	1399	114																							
22	21	2760	1650	4410	8835	33318	1874	1140																							
23	22	3110	1212	4322	8732	38302	2296	114																							
24	23	3190	938	4128	8450	42160	2642	114																							
25	24	3105	762	3867	7995	44271	2893	114																							
26	25	2930	388	3318	7185	46270	3025	114																							
27	26	2620	156	2776	6294	46214	3039	114																							
28	27	2270	62	2332	5108	45244	2946	114																							
29	28	1950	38	1988	4320	43872	2799	114																							
30	29	1620	18	1638	3626	41800	2620	114																							
31	30	1365	6	1371	3009	39869	2416	114																							
32	31	1130	0	1130	2501	37338	2212	114																							
33	32	960		960	2090	35204	2013	114																							
34	33	810		810	1770	32748	1825	114																							
35	34	680		680	1490	30827	1654	114																							
36	35	580		580	1260	28340	1495	114																							
37	36	500		500	1080	26630	1351																								
38	37	440		440	940	24665																									

STATE OF CONNECTICUT
WATER RESOURCES COMMISSION
State Office Building
Hartford, Connecticut

STATE WATER RESOURCES
COMMISSION
RECEIVED

JUL 19 1966

ANSWERED.....
REFERRED.....
FILED.....

APPLICATION FOR CONSTRUCTION PERMIT FOR DAM

For West Goshen Realty Association Inc.

Date _____

Address West GoshenConnecticut 06797

Tel. No. _____

Location of Structure:

Goshen, ConnecticutShown on USGS Quadrangle CornwallMarshepaug River

at one inches ~~west~~ of Lat. 41°-45'-30
north
and 0.7 inches ~~east~~ of Long. 73°-15'
west

Directions for reaching site from nearest village or route intersection:
(to sketch on reverse side)

Route 4 to West Goshen. South 2.8 miles on Beach St. to Ives Road.

West on Ives Road one mile to Marshepaug Road. Northwesternly on Marshepaug
Road 0.6 mile to dam site.

This is an application for: (New Construction) (Alteration) (Repair) (Removal)
(check one or more of above)

The pond is to be used for: Recreation at Housing DevelopmentDimensions of Pond: width 2500 feet length 7500 feet area 390 AcresAverage depth of water immediately above dam: 28 feetLength of dam: 1320 feetCrest of spillway: Crest 80 feetHeight of abutments above spillway: Crest 8 feetSpillway construction: Concrete Ogee with downstream chute.Type of dike construction: Rolled Earth Embankment

Spillway section will be set on: (Bedrock) (Gravel) (Clay) (Fill)
(check one or above)

Attachments: Detail Plans and Specifications presented with this

application.

Signed: West Goshen Realty Assoc

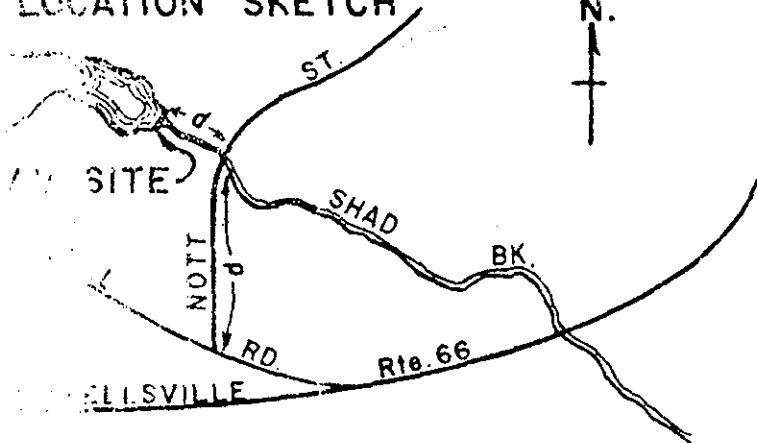
Alfred H. Wright Jr. Pres
Nelson J. Bennett, Vice Pres
Anderson-Nichols

Name of Engineer, if any B-27

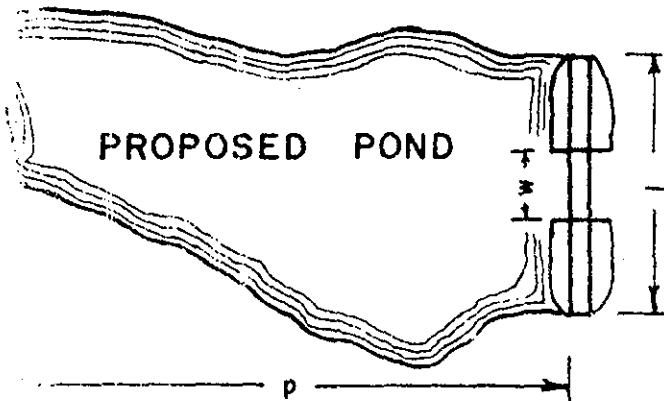
Show details of

SAMPLE DATA

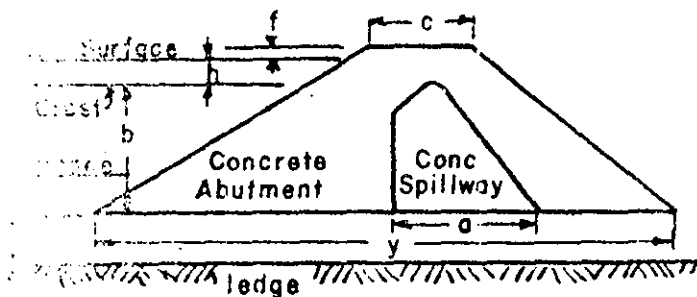
LOCATION SKETCH



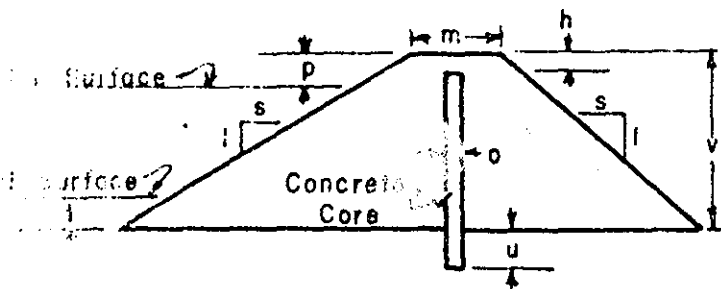
PLAN



SPILLWAY SECTION



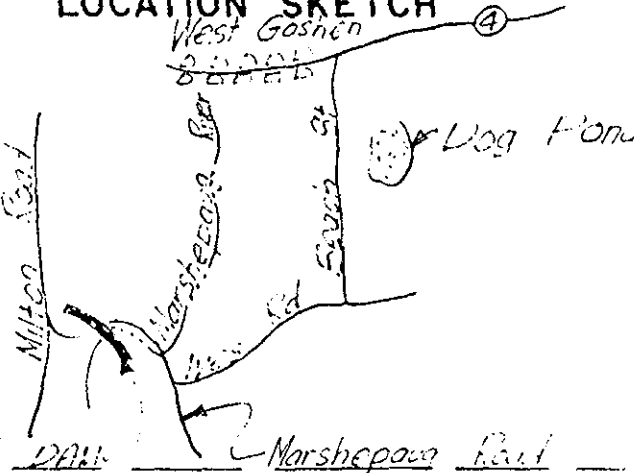
DIKE SECTION



APPLICANT'S DATA

Show only features of sample which are applicable and dimensions which reflect y

LOCATION SKETCH



SITE PLAN

SEE CONSTRUCTION PLANS
SUBMITTED HERewith

SPILLWAY SECTION

SEE CONSTRUCTION PLANS
SUBMITTED HERewith

NOTE...

If there are two methods of discharge S

DIKE SECTION

SEE CONSTRUCTION PLANS
SUBMITTED HERewith

STATE OF CONNECTICUT
STATE COMMISSION ON CONSTRUCTION
State Office Building, Hartford 15, Connecticut

CONSTRUCTION PERMIT FOR DAM

September 20, 1966

West Goshen Realty Association, Inc.
West Goshen, Connecticut

TOWN: Goshen
RIVER: East Branch Shepaug River
TRIBUTARY: Marshepaug River

Gentlemen:

Your application for a permit to (~~xxxxxx~~) a dam on Marshepaug River
(construct)

in the Town of Goshen in accordance
with plans prepared by Anderson-Nichols and Company, Inc.
dated July, 1966 has been reviewed.

The construction, in accordance with those plans, is APPROVED under the conditions which follow.

- I. The Commission shall be notified as follows:
 - A. When construction is started.
 - B. When foundation is excavated.
 - C. When project is completed and before water is impounded.
 - D. When project is completed and ready for final inspection.
- II. This permit with the plans and specifications must be kept at the site of the work and made available to the Commission at any time during the construction.
- III. If any changes are contemplated or required, the Commission must be notified and supplementary approval obtained.
- IV. If the construction authorized by this permit is not started within two years of the date of this permit and completed within four years of the same date, this permit must be renewed.
- V. Additional requirements -

E. D'APPOLONIA

CONSULTING ENGINEERS, INC.

May 27, 1969

15 DUFF ROAD
PITTSBURGH, PA. 15235

TELEPHONE
(412) 242-5107

Project No. 68-186

State of Connecticut
Water Resources Commission
State Office Building
Hartford, Connecticut 06115

STATE WATER RESOURCES
COMMISSION
RECEIVED

JUN 2 1969

Proposed Dam and Recreation Lake
West Goshen, Connecticut

ANSWERED _____
REFERRED _____
FILED _____

Gentlemen:

As discussed with your Mr. Pelletier in a recent telephone conversation, our client, Boise Cascade Properties, Inc., (formerly United States Land, Inc.) is presently negotiating the purchase of the land and the design for the proposed dam and recreation lake near West Goshen, Connecticut. The facility design was previously submitted to you by the West Goshen Realty Association, Inc. Your office issued an original dam construction permit in July, 1966 and reissued the permit on September 17, 1968. It is our understanding prior to proceeding with the dam construction, a reissue of the permit to Boise Cascade Properties, Inc., is required.

The present design of the dam has been discussed with Anderson-Nichols & Company, Inc., and with Haley & Aldrich, Inc., the original engineer and the original soils consultant. We concur with the existing design for the major portions of the dam and hydraulic structures as originally submitted to you by West Goshen Realty Association, Inc., through Anderson-Nichols & Company, Inc. There are, however, two relatively minor modifications which we feel should be incorporated into the design to assure the intended behavior of the dam structure. A description of the proposed changes and the reasons for making them are as follows:

1. Increase the thickness of upstream impervious blanket from two feet to three feet: The original design called for a two-foot-thick, impervious blanket upstream from the dam. Calculations indicate that the two-foot-thick blanket would be sufficient to maintain an acceptable amount of seepage under the dam and that an additional foot of thickness of blanket would do very little to reduce the flow. However, due to the nature of the borrow material at this site, particularly with respect to the numerous boulders nested within the silty sand matrix, we feel that it is prudent to add one additional foot to the blanket thickness to account for potential relatively pervious zones due to segregation of the borrow material.

E. D'APPOLONIA

CONSULTING ENGINEERS, INC.

State of Connecticut

-2-

May 27, 1969

2. Reduce the outlet pipe size from 36-inch diameter to 24-inch diameter: The primary purpose of the outlet pipe in the structure is to lower the lake level for shoreline repair work and for draining the lake completely if such a need would ever occur. The 36-inch-diameter pipe has the capability of drawing the lake level down at a rate of approximately one foot per day. Based on experience and published data, we feel that a stability analysis considering the rapid drawdown pore pressures is appropriate for this rate of drawdown. Such an analysis indicates that the upstream slope of the dam would have a safety factor less than 1.0 against a deep seated failure.

A 24-inch-diameter pipe will draw the lake down at a rate of 0.3 foot per day; allowing a much greater time for the pore pressures within the embankment material to dissipate. Such a reduction in pore pressures greatly increases the stability of the slope.

These proposed changes were submitted to Anderson-Nichols & Company, Inc., for their comments which are included in the attached copy of their May 7, 1969 letter. The major point raised by Anderson-Nichols is that concerning the relative adequacy of 24 and 36-inch outlet pipes for diversion of the stream during construction. However, calculations show that while either size pipe would be capable of discharging a normal stream flow of less than 20 cfs (as measured in the field), neither pipe would be capable of discharging flows associated with storms. The diversion scheme which we would suggest to the contractor is shown on the enclosed Drawing No. 68-186-SK1. The scheme is based on constructing a small temporary dike across the narrow portion of the valley, 400 feet upstream from the dam to pond water while the central portion of the main embankment is being filled to a safe height above the valley bottom. This type of arrangement is particularly attractive at the proposed site because the very flat topography in the reservoir area provides a large storage volume for a small rise in water level. A 14-foot high dike to elevation 1130 provides more than 40 days of storage based on a conservative normal flow of 30 cubic feet per second. This amount of storage will provide the required safety against damage to the contractor's operation due to runoff during the closing period for the dam. We would further suggest to the contractor that a corrugated metal pipe be provided through the temporary dike to allow normal flows to pass down to the main outlet works after the central portion of the dam has been closed.

E. D'APPOLONIA

CONSULTING ENGINEERS, INC.

State of Connecticut

-3-

May 27, 1969

Due to contractual agreements, it will not be possible to formally apply for renewal of the construction permit until the negotiations between Boise Cascade Properties, Inc. and West Goshen Realty Association, Inc. have been finalized. However, we would appreciate a letter from your office indicating the final procedure required for renewal as well as your approval of the modifications discussed above. At the time of formal application, the enclosed Drawings Nos. 68-186-E1 and E2 showing the above discussed modifications will be submitted as supplemental drawings to the Anderson-Nichols and Company original drawings. An appendix to the specifications will also be submitted to reflect these modifications and to incorporate general contractual conditions perferred by Boise Cascade Properties, Inc.

Thank you for your interest in this project. Please let us know if we may provide further information.

Very truly yours,

Richard D. Ellison

Richard D. Ellison

RDE:dhc

Enclosures

cc: Mr. Herman J. Kropper (enclosures)
Mr. James R. Rogers (enclosures)
Mr. Roger Hussey (enclosures)

E. D'APPOLONIA

CONSULTING ENGINEERS, INC.

September 23, 1969

15 DUFF ROAD
PITTSBURGH, PA. 15235

TELEPHONE
(412) 242-51

STATE WATER RESOURCES
COMMISSION
RECEIVED

Project No. 68-186

SEP 24 1969

Mr. H. Robert Hoffman,
Macchi & Hoffman
44 Gillette Street
Hartford, Connecticut 06105

ANSWERED _____
REFERRED _____
FILED _____

Diversion Dike

Dear Mr. Hoffman:

Depending on stream flow conditions and predicted weather conditions at the Woodridge Dam site, a temporary diversion dike may be required during the three-to-seven-day period in which the earth closure at the creek is being made. We have considered the following conditions for establishing the required dike height:

1. Twice the normal stream flow for 30 days, plus two, two-year recurrence storms with no outflow for this period.
2. Twice the normal stream flow for 30 days, plus one 10-year recurrence storm with no outflow for this period.
3. Twice the normal stream flow for 25 days, plus one two-year recurrence storm and one 10-year recurrence storm with no outflow during this period.

The normal stream flow used in the calculations are twice those corresponding to the stream gaging station No. 2019.3 located on the Marshespaug River, 500 feet downstream from the dam. The maximum water rise behind the dike for the above cases was to elevation 1126. Therefore, it is concluded that the maximum height of dike required is 12 feet with the lowest portion of its base at elevation 1116.

E. D'APPOLONIA

CONSULTING ENGINEERS, INC.

Mr. H. Robert Hoffman

-2-

September 23, 1969

Initially, we had intended to locate the dike about 900 feet upstream from the dam and to completely stop the stream flow until the dam closure height had reached elevation 1126. However, inasmuch as the outlet pipe has been installed and both sides of the dam have been filled above elevation 1126, we are planning to close the dam following the procedure outlined below:

1. A dike will be constructed in the location shown on the attached Drawing 68-186-SK3, and the normal stream flow will pass into the permanent outlet pipe through the dam. The dike will be constructed of compacted fill with 2.5 to 1 slopes, both upstream and downstream.
2. Since the permanent outlet pipe is capable of passing the normal stream flow (less than 10 cfs) with a water rise to only elevation 1118, the dike height may be reduced to about six feet or to elevation 1120 instead of elevation 1126 as previously discussed. The final dike height will be determined based on the flow in the stream and the projected weather forecast at the time of closure. It is expected that less than three days will be required to raise the entire dam embankment closure above elevation 1126.

I have enclosed a set of our diversion design calculations for your reference. Our present plan is to begin the closure during the first week in October 1969. I suggest that you contact Mr. Michael Taylor at the site (telephone (203) 482-3160) prior to October 1, 1969 to discuss our final diversion plan.

Very truly yours,

Richard D. Ellison

Richard D. Ellison

RDE:pao
Enclosure

cc: Mr. William H. O'Brien, III
Mr. J. W. Ford
Mr. Roger Hussey
Mr. Michael Taylor

E. D'APPOLONIA
CONSULTING ENGINEERS, INC.

15 DUFF ROAD
PITTSBURGH, PA. 15235

November 18, 1969

TELEPHO
(412) 242-5

Project No. 68-186

STATE WATER RESOURCES
COMMISSION
RECEIVED

NOV 21 1969

Mr. H. Robert Hoffman
Macci and Hoffman
44 Gillette Street
Hartford, Connecticut 06105

ANSWERED _____
REFERRED _____
FILED _____

Dear Mr. Hoffman:

Enclosed are design calculations and sketches of a low-flow augmentation system to be installed at Boise Cascade's Woodridge Lake Dam, West Goshen, Connecticut. (See *Form 5 6/1/70*)

At the request of the City of Waterbury and a fishing club downstream of the dam, Boise Cascade Properties, Inc. has agreed to install a system with the capability of drawing water from mid-depth of the lake to augment low-flow conditions in the Marshepaug River by at least 2 cfs. The system as designed will have a capacity of at least 2.5 cfs throughout its life.

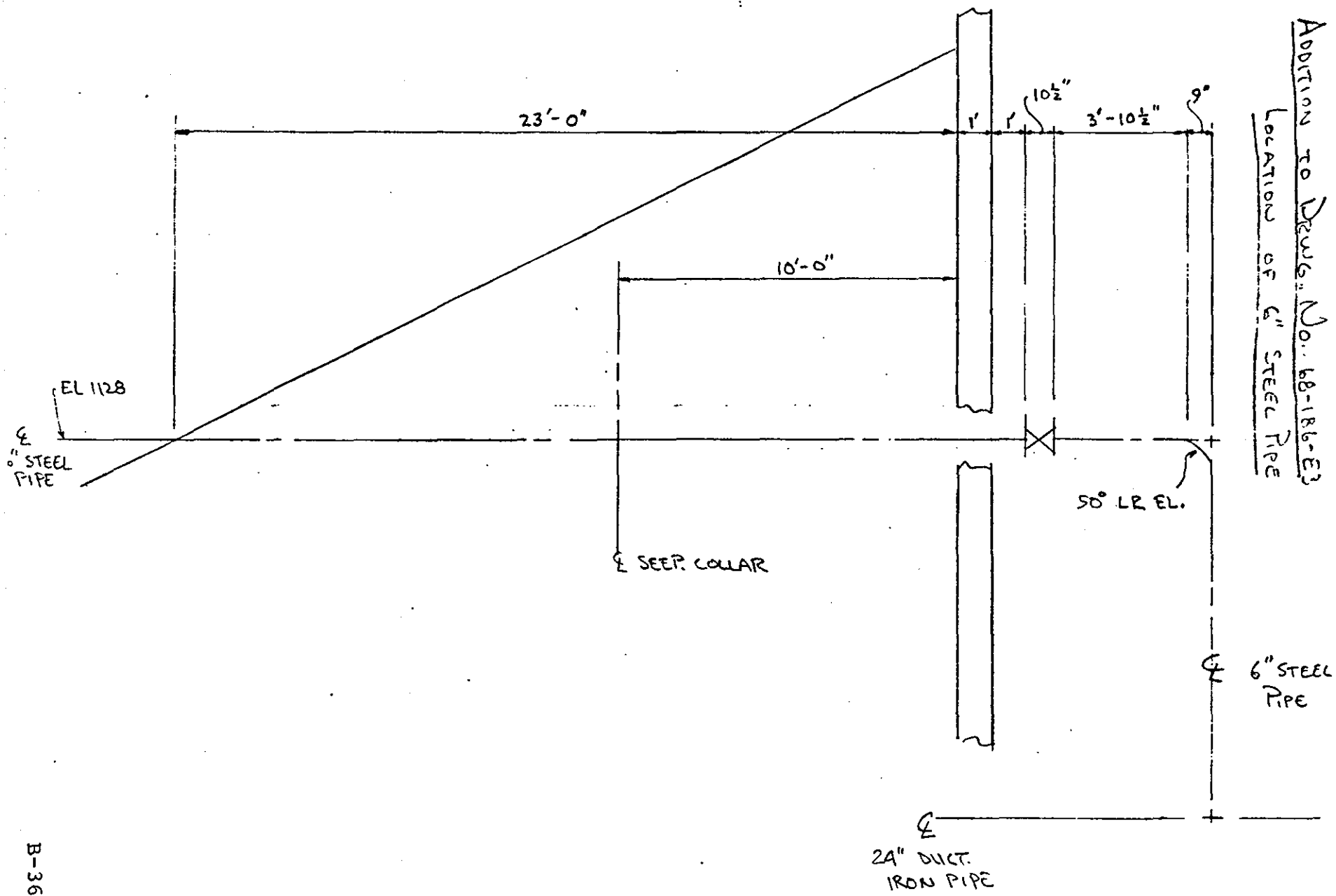
Very truly yours,

Richard D. Ellison
Richard D. Ellison

RDE:isw
Encl.

cc: Mr. Roger Hussey
Mr. J. W. Ford
Mr. Wm. H. O'Brien, III ✓

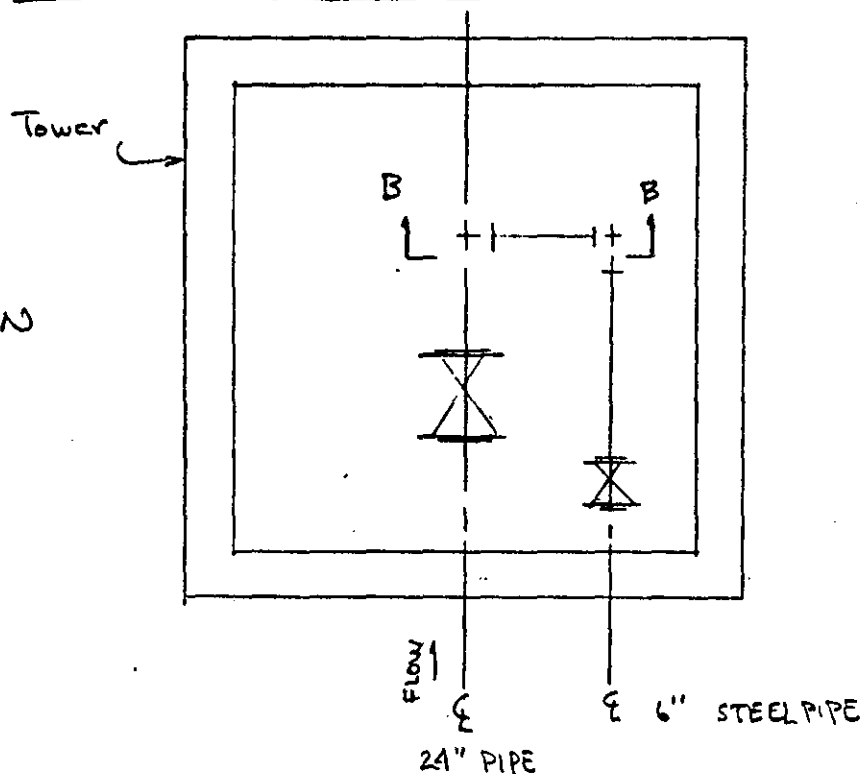
ADDITION TO DEWS. NO. 68-186-E3
LOCATION OF 6" STEEL PIPE



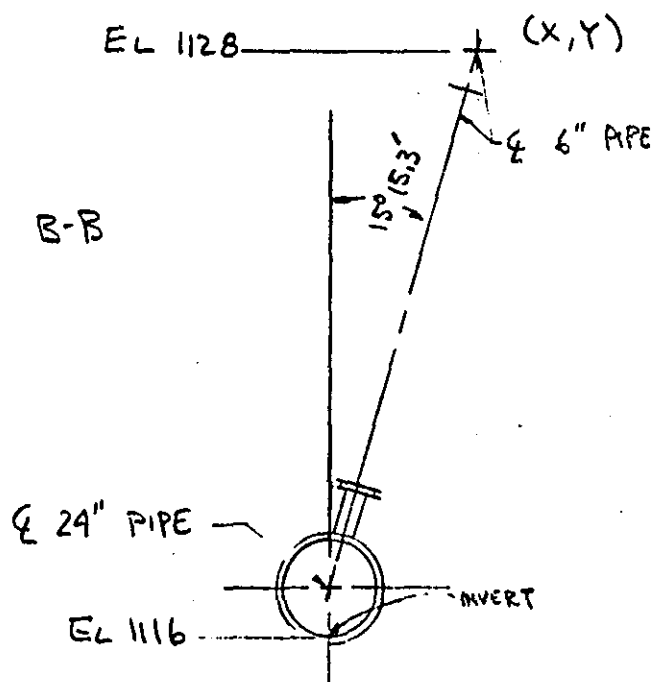
ADDITION TO DRWG. NO. 68-146-E3

LOCATION OF 6" STEEL PIPE

PLAN



SECTION B-B

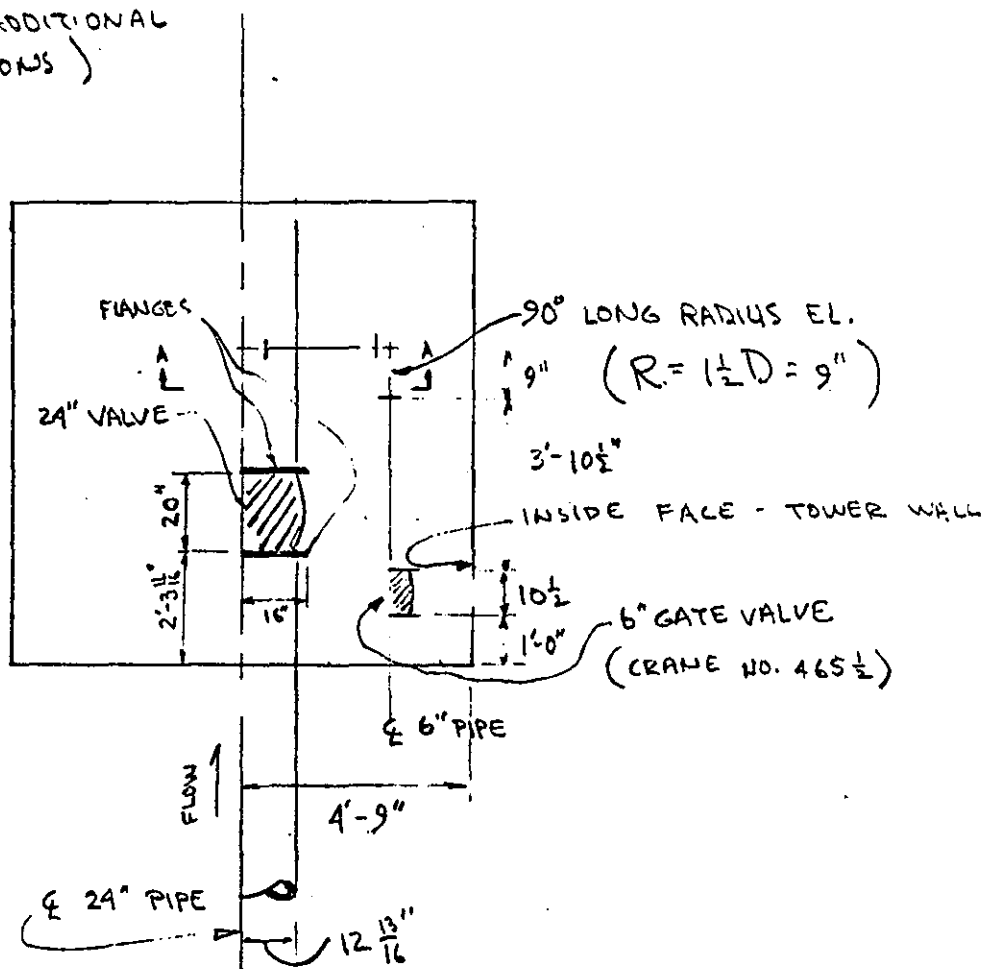


X-Y COORD.S FROM
INTERSECTION OF
24" PIPE & WITH
UPSTREAM TOWER
WALL TO INTERSECTION
OF & OF 6" PIPE
WITH SAME WALL:

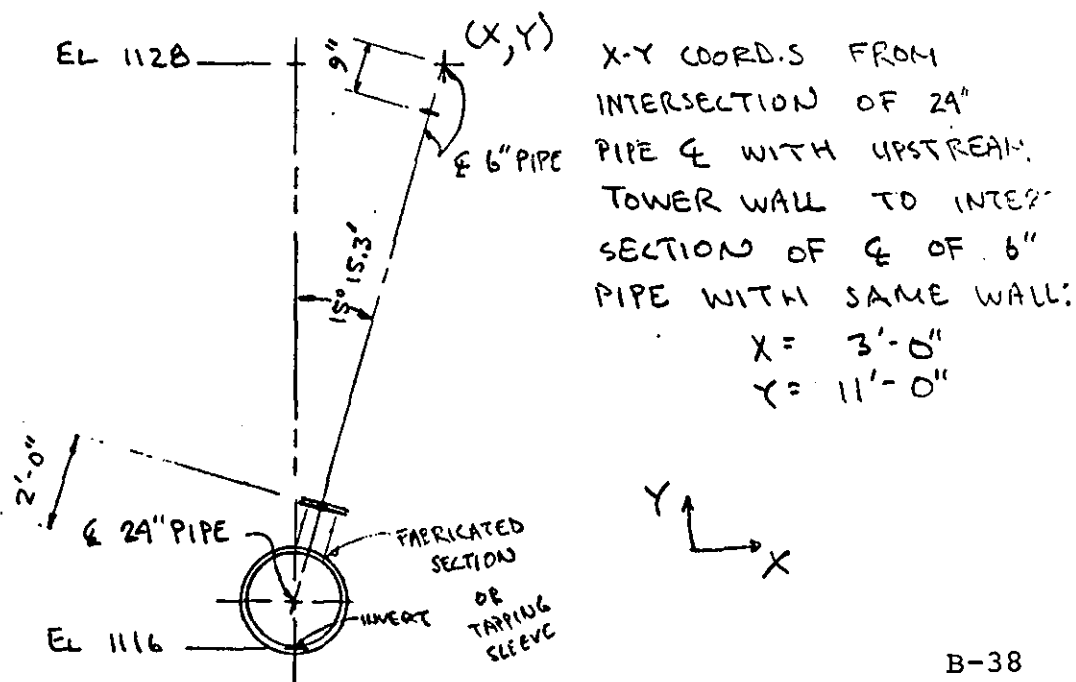
$$X = 3'-0"$$

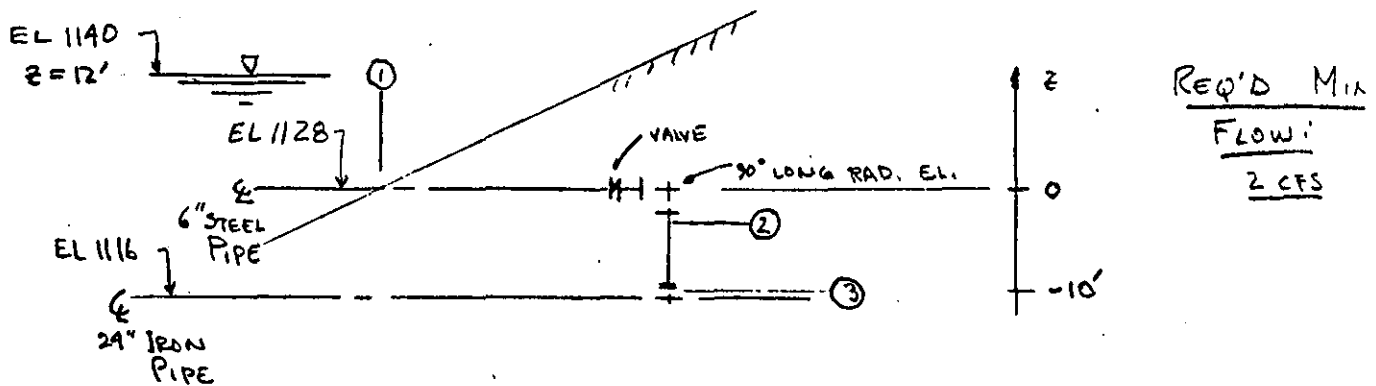
$$Y = 11'-0"$$

PLAN



SECTION A-A



HYDRAULIC CALC'S

From Mass Conservation: $v_2 = v_3 \equiv v$

From Bernoulli E_z :

$$\left[\frac{p}{\gamma} \right]_1 = 12'$$

$$\left[\frac{p}{\gamma} + \frac{v^2}{2g} + z \right]_2 = \left[\frac{p}{\gamma} \right]_1 - \frac{v^2}{2g} [k_{ent.} + k_{valve} + k_{elbow} + k_L]$$

$$\left[\frac{p}{\gamma} + \frac{v^2}{2g} + z \right]_3 = \left[\frac{p}{\gamma} \right]_1 - \frac{v^2}{2g} [k_{ent.} + k_{valve} + k_{elbow} + k_L + k_{exit}]$$

where

p = pressure

γ = unit wt. of water

k the k 's are loss coef.s

Assume 24" PIPE NOT RUNNING FULL.

HENCE

$$\left[\frac{p}{\gamma} \right] = 0$$

HYDRAULIC CALC'S

6" EXTRA STRONG STEEL PIPE

$$D = I.D. = 5.761 \text{ in} = 0.480 \text{ ft.}$$

$$A = \text{Area} = \frac{\pi}{4} (5.761)^2 \times \frac{1}{144} = 0.181 \text{ ft}^2$$

length of straight sections to elbow:

$$L = 28.87 \text{ ft} \quad \frac{L}{D} = 60.1 \quad \frac{D}{k} = \frac{.48}{.25 \times 10^{-3}} = 1920$$

say 2000

length of str. sections to 24" pipe:

$$L = 37.52 \text{ ft.} \quad \frac{L}{D} = 78.1 \quad \frac{D}{k} = 2000$$

For long radius elbow ($\frac{R}{D} = 1.5$), take

$$k_{elb.} = 0.28 \quad \text{See Rouse, Eng. Hydraulics, p. 421}$$

Also, take

$$k_{entrance} = 0.5 \quad (\text{sharp edged})$$

$$k_{valve} = 0.19 \quad (\text{fully opened gate valve})$$

$$k_{exitsection} = 0.5$$

f

$$k_L = f \frac{L}{D}$$

HYDRAULIC CALC.S

Assuming pressure is also zero at exit from elbow :

$$\frac{V^2}{2g} = 12 - \frac{V^2}{2g} (0.5 + 0.19 + 0.28 + 60.1 f)$$

or

$$V^2 = \frac{12(64.4)}{1.97 + 60.1f} = \frac{772}{1.97 + 60.1f}$$

Try $f = 0.0178$

$$V^2 = \frac{772}{3.04} = 254$$

$$V = 15.9 \text{ ft sec}^{-1}$$

$$\frac{VD}{\nu} = \frac{15.9(48)}{1 \times 10^{-5}} = 7.64 \times 10^5 \quad f = 0.0174 \quad \text{say } \underline{ok}$$

$$Q = 2.88 \text{ cfs} \quad \underline{ok}$$

Assuming near-vertical length runs full :

$$\frac{V^2}{2g} - 10 = 12 - \frac{V^2}{2g} (0.5 + 0.19 + 0.28 + 0.5 + 78.1 f)$$

or

$$V^2 = \frac{22(64.4)}{2.47 + 78.1f} = \frac{1420}{2.47 + 78.1f}$$

Try $f = 0.017$

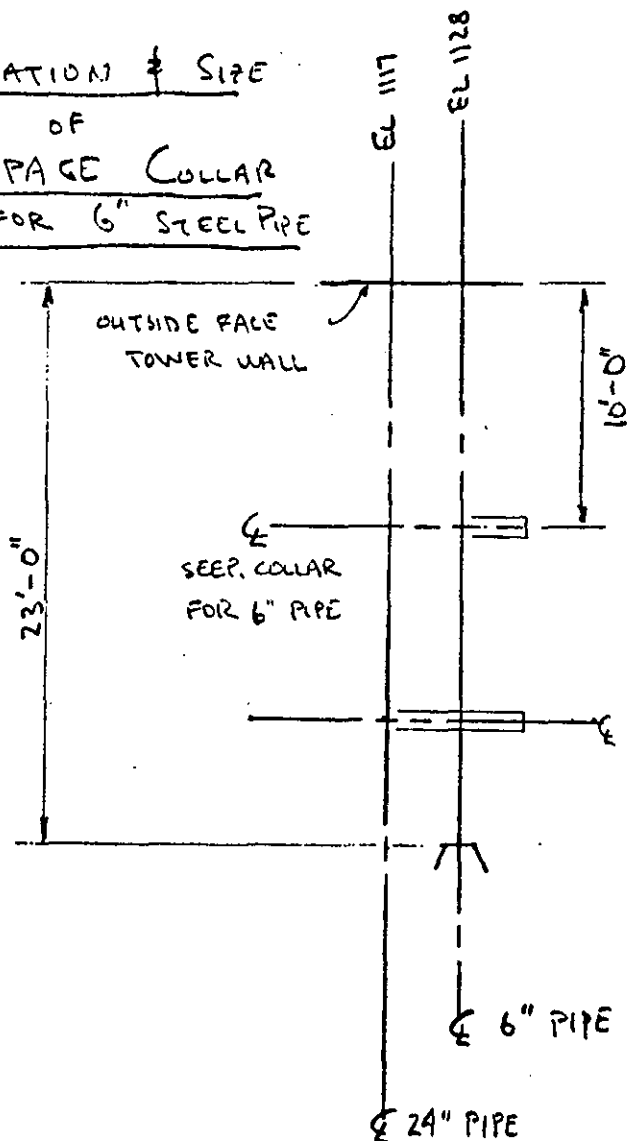
$$V^2 = \frac{1420}{3.80} = 374$$

$$V = 19.3 \text{ ft sec}^{-1}$$

$$\frac{VD}{\nu} = 9.27 \times 10^5 \quad f = 0.0172 \quad \text{say } \underline{ok}$$

$$Q = 3.49 \text{ cfs} \quad \underline{ok}$$

LOCATION & SIZE
 OF
 SEEPAGE COLLAR
 FOR 6" STEEL PIPE



6" PIPE

ORIG. SEEPAGE LENGTH, L

$$L = 23' + 11.5' = 34.5'$$

FOR A 5.5' x 5.5' x .75' SEEP. COLLAR,

& A 2' x 2' CONCRETE CASING

FOR PIPE:

SEEPAGE LENGTH, L_s

$$L_s = 34.5 + 2 \left[\frac{1}{2} (5.5 - 2) (.75) \right] + 1.25 = 40.75$$

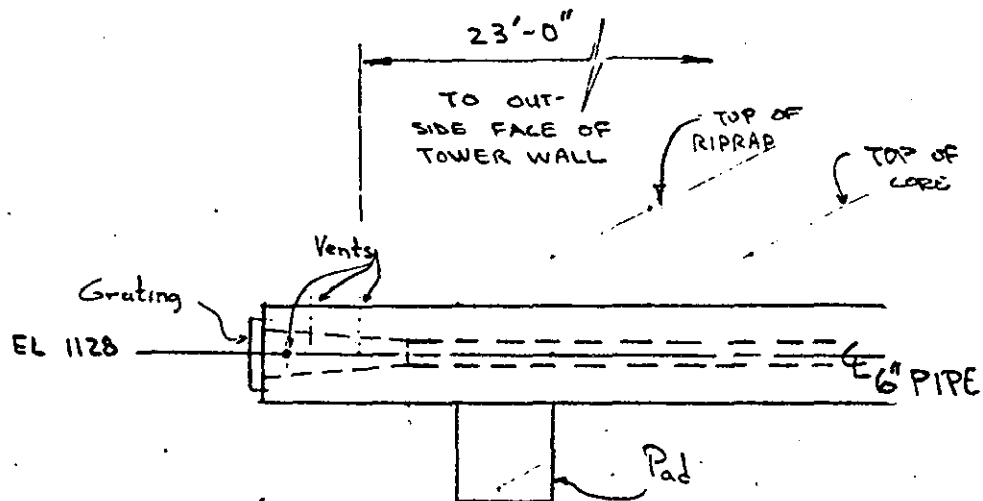
$$\frac{L_s}{L} = \frac{40.75}{34.5} = 1.18 > 1.15$$

OK

(SEE DEPT. OF AGR
 ENG. MEMO. -27
 P. E-5)

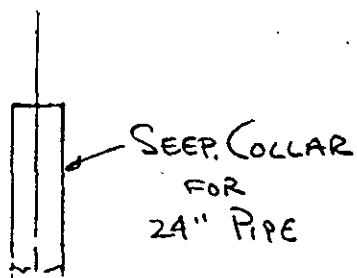
Note: Final design
 of seep. collar:
 6' x 6' x 9"

(See calc. s)
 11/1/69



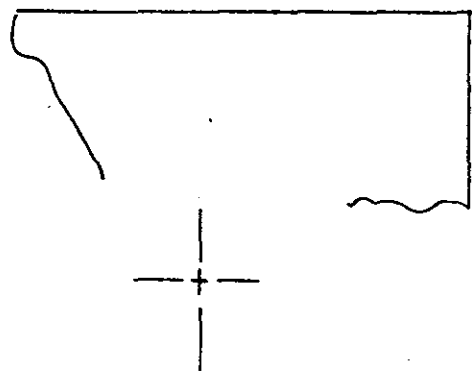
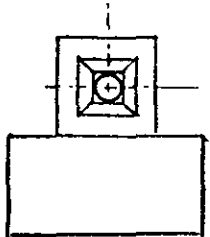
ave. frost
penetration
(25")

SCALE: 1" = 4'-0"



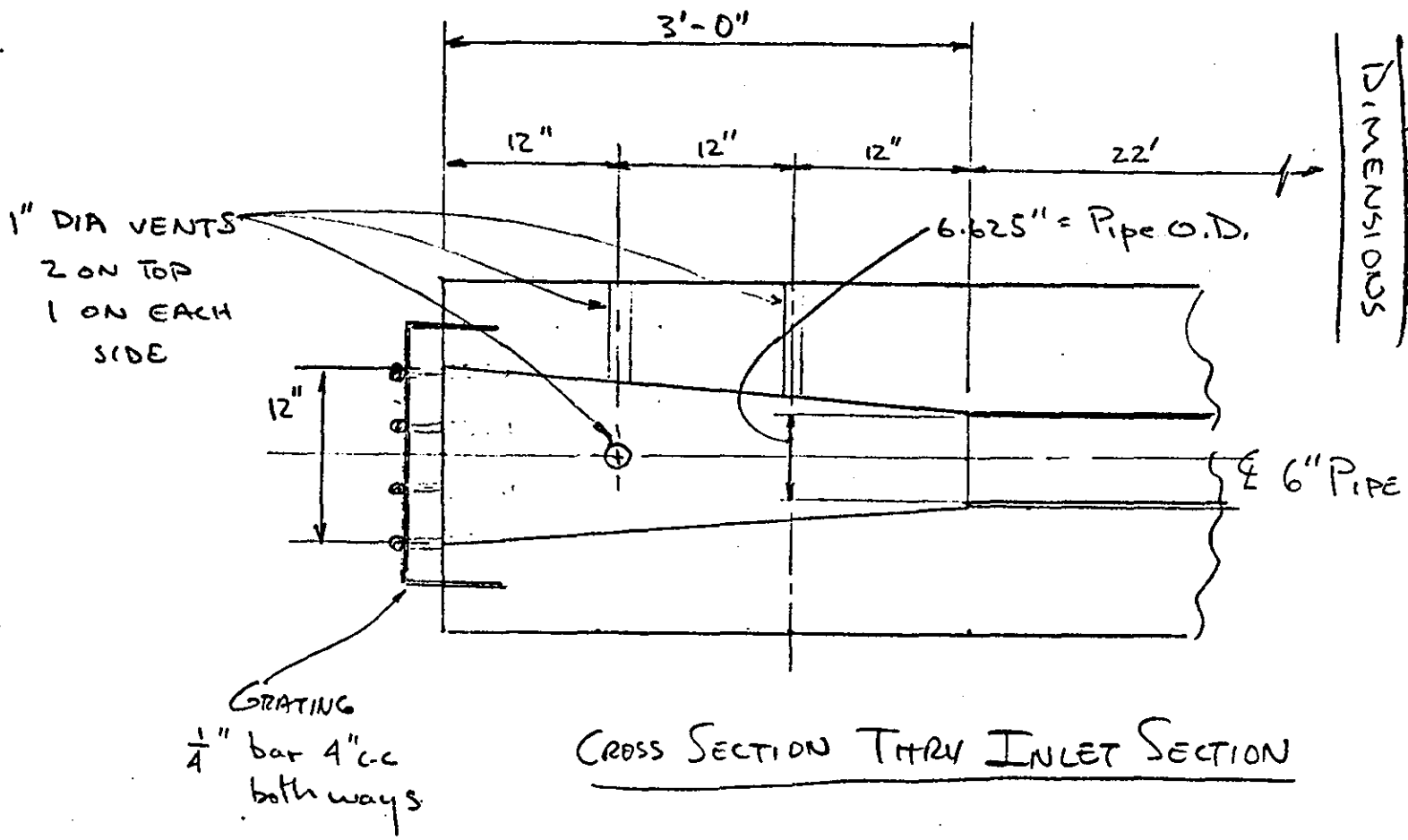
24" PIPE

INTAKE
STRUCT. DIM'S
FINAL DESIGN



E. D'ARPOLONIA CONSULTING ENGINEERS, INC.
 BY DLR DATE 11/1/69 SUBJECT WEST GOSHEN SHEET NO 15
 CHKD. BY CC DATE 11/13/69 Flow Area 545 PROJ. NO. 62-15

INLET SECTION
DIMENSIONS



CROSS SECTION THRU INLET SECTION

STRUCT. DESIGN

LOADSFactors

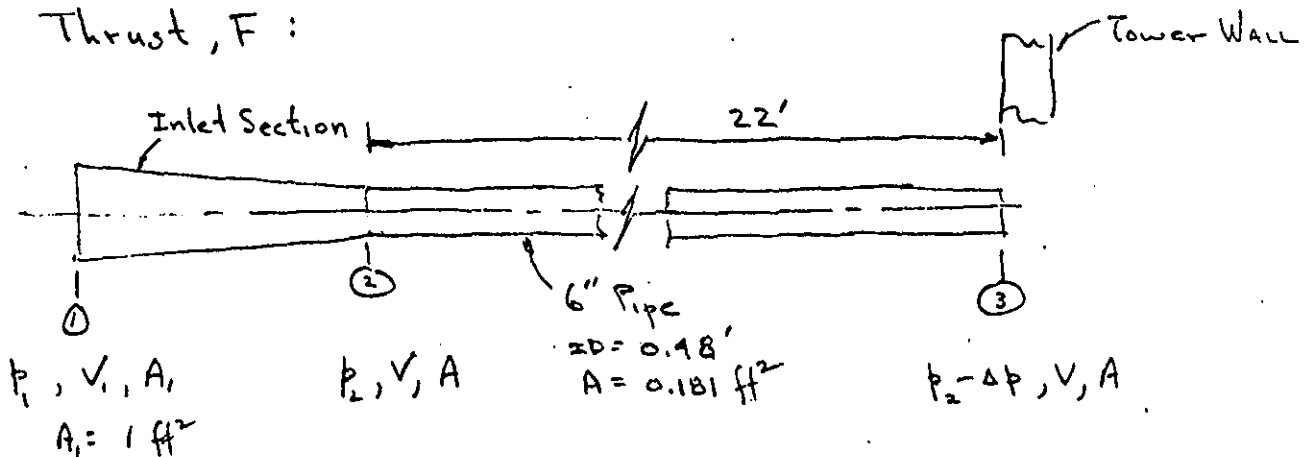
Hydraulic pressures & thrusts
 Overburden & soil reaction
 Settlement
 Frost heave in event of lake drawdown
 Temp. changes within Intake Tower
 Structure's own weight

Concrete Covered Pipe Section

Hydrostatic Pressure: Max Head = 14.5 ft.

$$\text{Pressure} = (14.5)(62.4) = 905 \frac{\text{lb}}{\text{ft}^2} = 6.28 \text{ psi} - \text{Insignificant}$$

Thrust, F:



From Hydraulic Calc.s, $Q = 3.49 \text{ cfs}$, $V = 19.3 \text{ fps}$,

$$V^2 = 374 (\text{fps})^2, \quad f = 0.017.$$

$$\text{Mass Conservation: } V_1 = \frac{VA}{A_1} = 3.49 \text{ fps}$$

Inlet Section
Neglecting Friction:

Momentum Eq.:

$$-F_T + p_1 A_1 - p_2 A_2 = \rho Q (V_2 - V_1)$$

$$= \rho (55.1 \frac{\text{ft}^3}{\text{sec}})$$

where F_T = thrust
on converging
sections

Bernoulli Eq.s:

$$\frac{p_1}{\gamma} + \frac{V_1^2}{2g} = 12' \quad ; \quad \frac{p_1}{\gamma} = 12 - \frac{V_1^2}{2g} = 11.8'$$

$$\frac{p_2}{\gamma} + \frac{V_2^2}{2g} = 12 \quad ; \quad \frac{p_2}{\gamma} = 12 - \frac{37.4}{64.4} = 12 - 5.8 = 6.2'$$

Substituting into momentum eq.:

$$-F_T + \gamma (11.8)(1) - \gamma (6.2)(1.81) = \frac{\gamma}{g} (55.1)$$

$$F_T = 559 \text{ lbs}$$

Straight Pipe Section

$$F_f = \gamma (\text{perimeter}) L = \Delta p A$$

$$\Delta p = f \frac{1}{2} \rho V^2 \frac{L}{D}$$

so

$$F_f = f \frac{1}{2} \rho V^2 \frac{L}{D} A$$

$$= (.017) \frac{\gamma}{2g} (37.4) \frac{22}{.48} (1.81)$$

$$F_f = 51.5 \text{ lbs}$$

where F_f = thrust due
to friction in
22' length

Total Thrust, $F = F_1 + F_2 = 610.5 \text{ lb}$ Insignificant

Hydraulic Forces due to seepage: Insignificant

Overburden, Soil Reaction, Settlement & Frost Heave

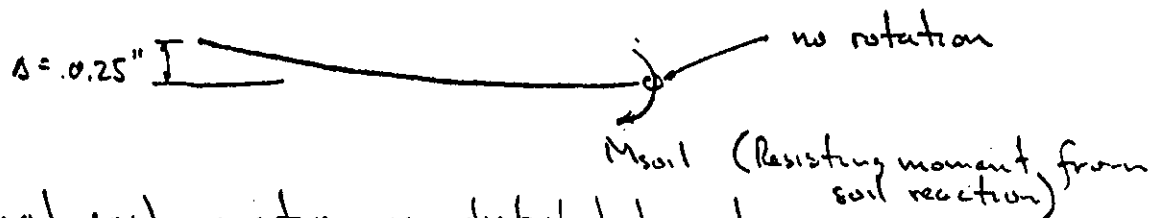
Since the overburden depth varies from zero to at most 12', overburden pressures will be of no consequence with regard to crushing of the conduit.

The soil is compacted & a differential settlement of no more than $\frac{1}{4}"$ is expected.

Average frost penetration is 25" (Water Atlas of U.S.).

Assuming a water expansion of 10% upon freezing & a 25% water content by volume, then the approximate frost heave displacement at surface of core is $\frac{1}{4}(0.1)(25) = 0.625"$. However, the conduit is not resting on the core surface (only protrudes through it) & the displacement of the conduit end due to frost heave is expected to be no more than $\frac{1}{3}$ of the displacement at the surface of the core, i.e., $\frac{1}{3}(0.625) = 0.208"$.

Thus the worst loading condition is a deflection (say up or down) of $\frac{1}{4}"$. Assuming the conduit is free at the pad & pinned at the tower wall, the loading is taken to be:



Actual soil reaction is distributed along conduit length & results in a lower max moment than the case assumed above.

Hence, P required for deflection Δ :

$$P = \Delta \frac{3EI}{L^3}$$

$$\text{Max Shear, } V = P = \frac{.25}{12} \frac{3}{10.6 \times 10^3} \frac{1}{144} EI = 4.08 \times 10^{-8} EI \quad \left(\begin{array}{l} V \text{ in lbs} \\ \text{for } EI \text{ in} \\ \text{lb in}^2 \end{array} \right)$$

$$\text{Max } M = M_{\text{support}} = PL = \frac{\Delta 3EI}{L^2} = \frac{.25}{12} \frac{3}{484} \frac{1}{144} EI$$

$$= 9.0 \times 10^{-7} EI \quad \left(\begin{array}{l} M \text{ in ft lb} \\ \text{for } EI \text{ in lb in}^2 \end{array} \right)$$

$$\text{Taking } E = 3 \times 10^6 \text{ psi}$$

$$\& I = \frac{(24)^4}{12} = 2.78 \times 10^4 \text{ in}^4$$

$$EI = 8.28 \times 10^{10}$$

&

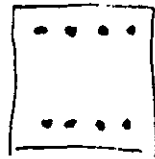
$$V = 3.39 \times 10^3 \text{ lb}$$

$$M = 74.5 \times 10^3 \text{ ft lb}$$

Design Reinf. for the above M & V .

Conduit Reinforcement

Neglect the pipe & consider one row each of tension & compression steel:



Take $d' = 3'' \neq d = 21''$

$$bd = 24(21) = 504 \text{ in}^2$$

$$\frac{d'}{d} = \frac{1}{7} = 0.143$$

$$M = 74.5 \text{ ft kips}$$

$$V = 3.39 \text{ kips}$$

From Concrete Design Handbook, p. 23:

Assume $A_s = A_s' = 1.2 \text{ in}^2$ (6-#4's, top & bottom)
 20,000 psi steel
 & 3,000 psi concrete ($n = 9.2$)

$$m = \frac{n A_s}{bd} + \frac{(2n-1) A_s'}{bd} = \frac{1.2}{504} (n + 2n-1) = \frac{1.2}{504} (26.6) = 0.0632$$

$$\begin{aligned}
 g &= \frac{1.2}{504} \left[n + (2n-1) \frac{d'}{d} \right] = \frac{1.2}{504} \left[9.2 + 17.4(0.143) \right] = \frac{1.2}{504} [9.2 + 2.49] \\
 &= \frac{(1.2)(11.69)}{504} = 0.02785
 \end{aligned}$$

Table 11: $k \approx 0.192$

$$\frac{1}{k} \frac{(2n-1) A_s'}{bd} = \frac{1}{0.192} \frac{(17.4)(1.2)}{504} = 0.216$$

$$\frac{1}{k} \frac{d'}{d} = \frac{1}{0.192} (0.143) = 0.745$$

Table 12: $z \approx 0.38$

So Table 13: $j \approx 0.92$

$$f_s = \frac{12,000 (74.5)}{(0.92)(21)(1.2)} = 38,600 \text{ psi}$$

$$f_c = \frac{f_s}{n} \frac{k}{1-k} = \frac{3.86 \times 10^4}{9.2} \frac{.192}{.808} = 1000 \text{ psi} \quad \underline{ok}$$

$$f_s' = 2f_s \frac{k - \frac{d'}{2}}{1-k} = 2(3.86 \times 10^4) \frac{(.049)}{.808} = 4,700 \text{ psi} \quad \underline{ok}$$

Allow steel to yield -
 then say f_s ok since reinf. will be placed
 all around $\frac{1}{4}$ since the steel pipe has been neglected.

Shear Stress:

$$v = \frac{V}{bd} = \frac{3.39 \times 10^3}{504} = 6.72 \text{ psi} \quad \underline{ok}$$

No stirrups required

for shear.



However, ACI Section 806 requires stirrups when compression steel is used.

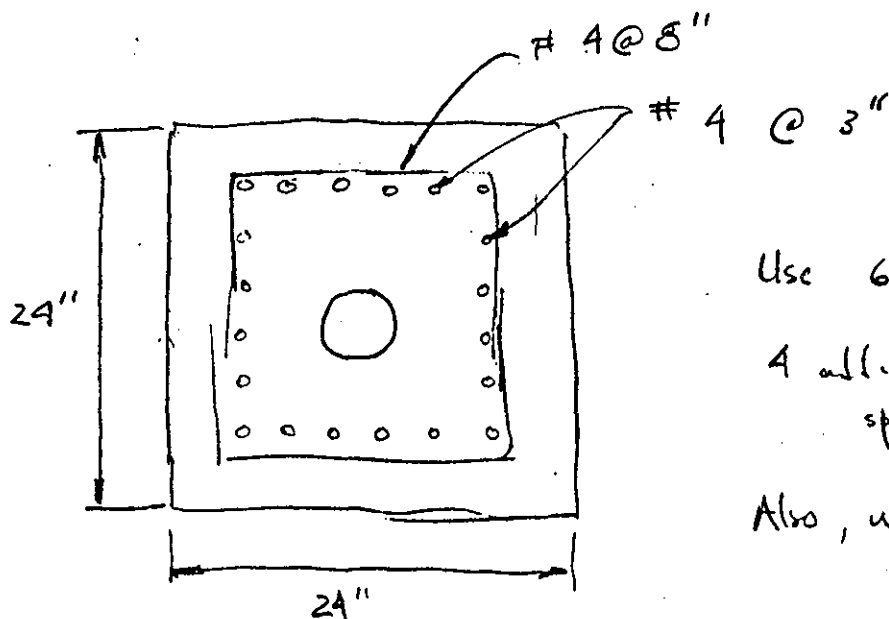
Could use

#6's @ 12" c-c

or

#4's @ 8" c-c

ACI Section 808 (a) requires 3" of concrete cover for concrete poured against the ground; 1½" for concrete exposed to weather (assuming #4 bars are used)



Use 6 #4's top & bottom

4 additional #4's equally spaced on sides

Also, use #4 stirrups @ 8" centers

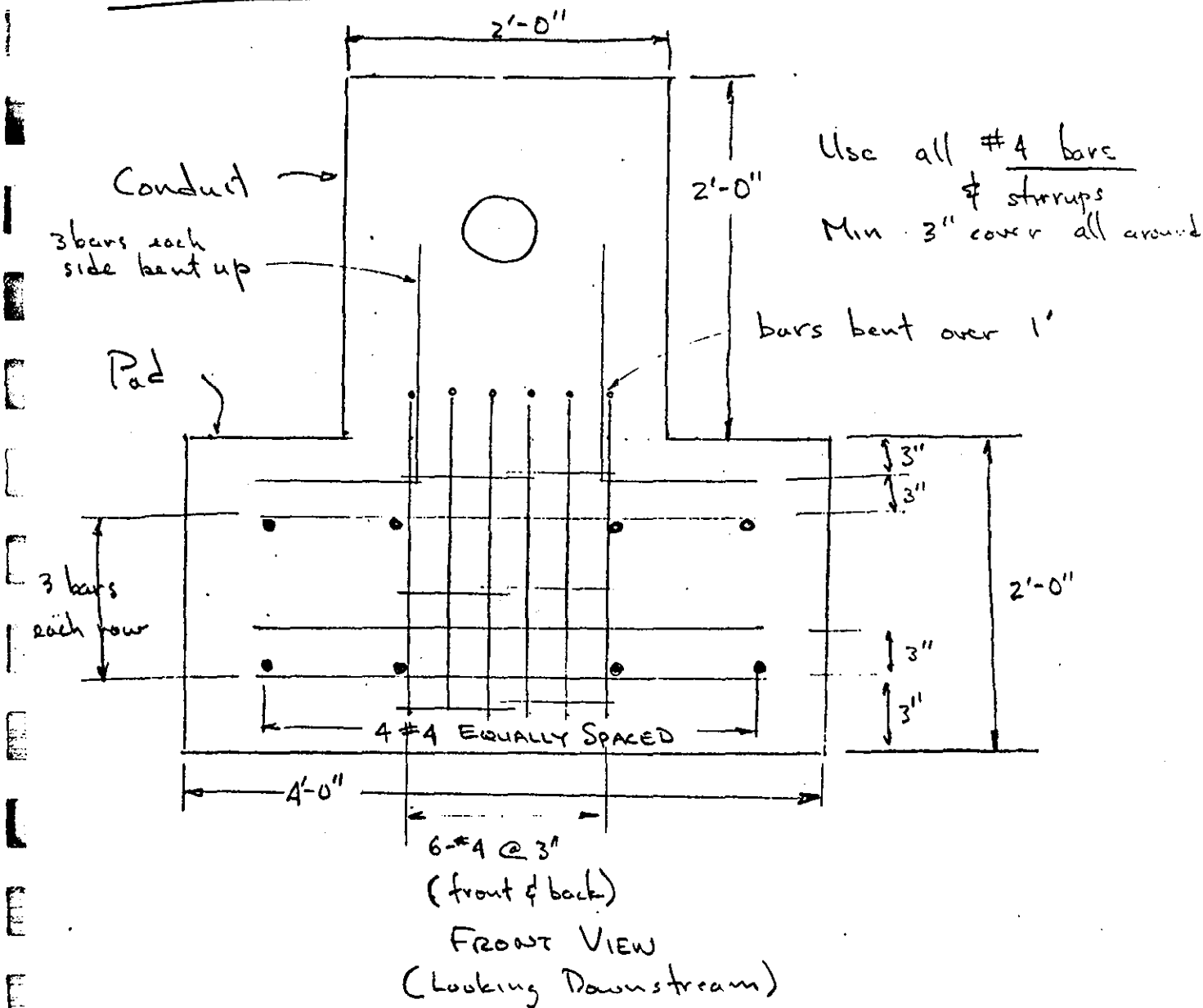
Cover: 3½"

Bond

$$u = \frac{V}{\Sigma_o j d} \quad ; \quad \Sigma_o = 6 (1.571) = 9.42$$

$$j d = (0.92) (21) = 19.3$$

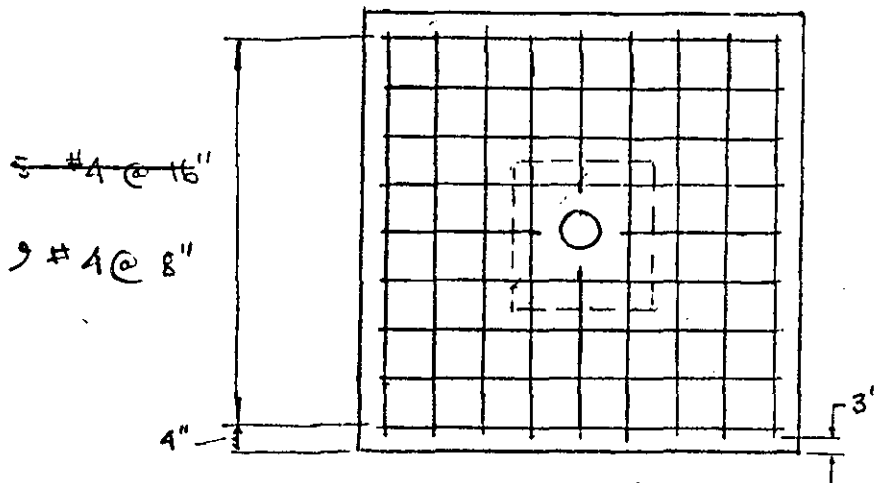
$$u = \frac{1.69 \times 10^3}{(9.42) (19.3)} = 9.3 \text{ psi} \quad \underline{\text{ok}}$$

PAD REINFORCING

Pad reinf. chosen so as to transmit design moment.

SEEPAGE COLLAR REINF.

Make collar 6' square x 9" thick.

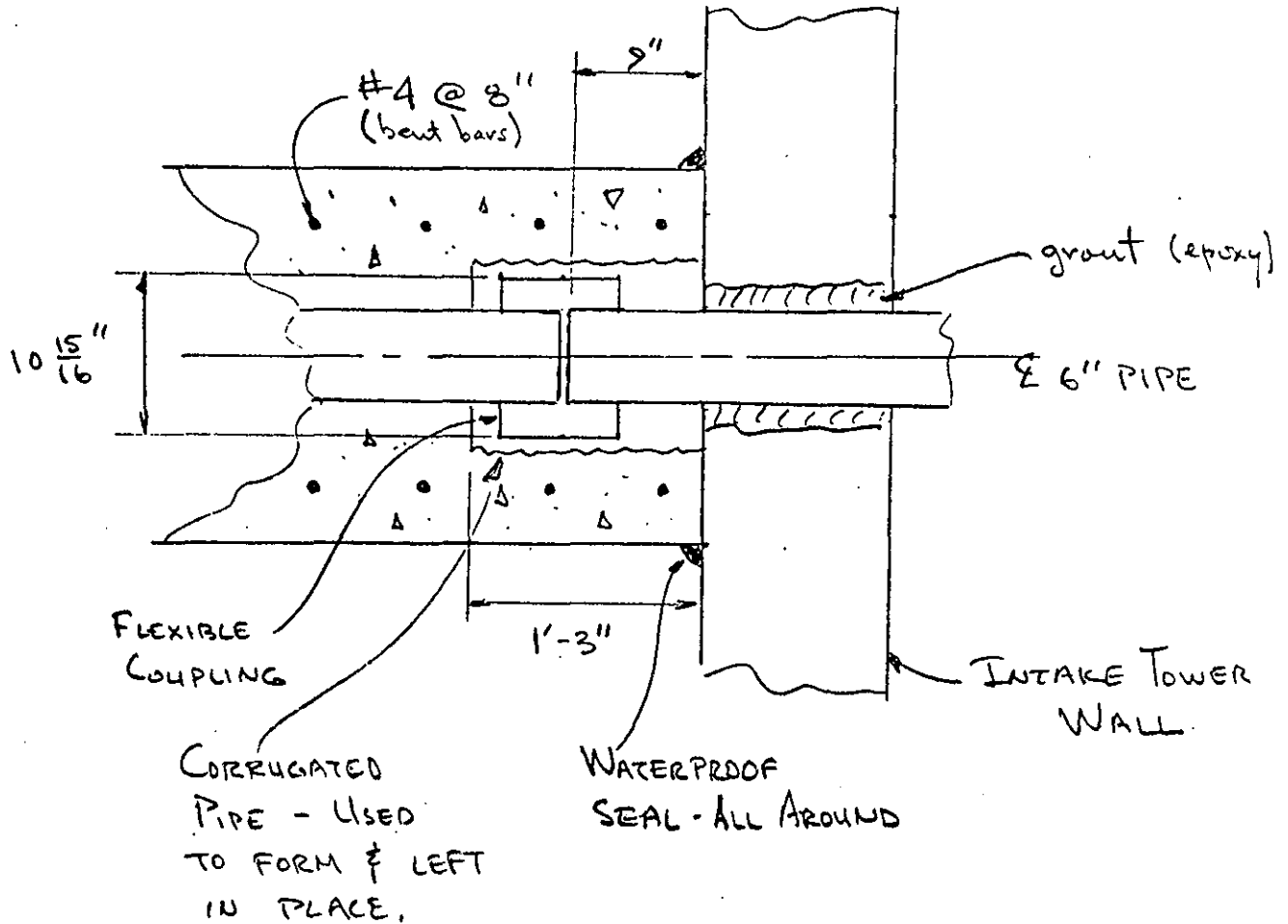


Temp. Reinf. :

Try 5 #4 $p = \frac{5(.2)}{6(144)(.75)} = 0.00154 < 0.0025 \text{ w.g.}$

Try 9 #4 $p = \frac{9(.2)}{6(144)(.75)} = 0.0027 > 0.0025 \text{ o.k.}$

CONSTRUCTION AT TOWER WALL



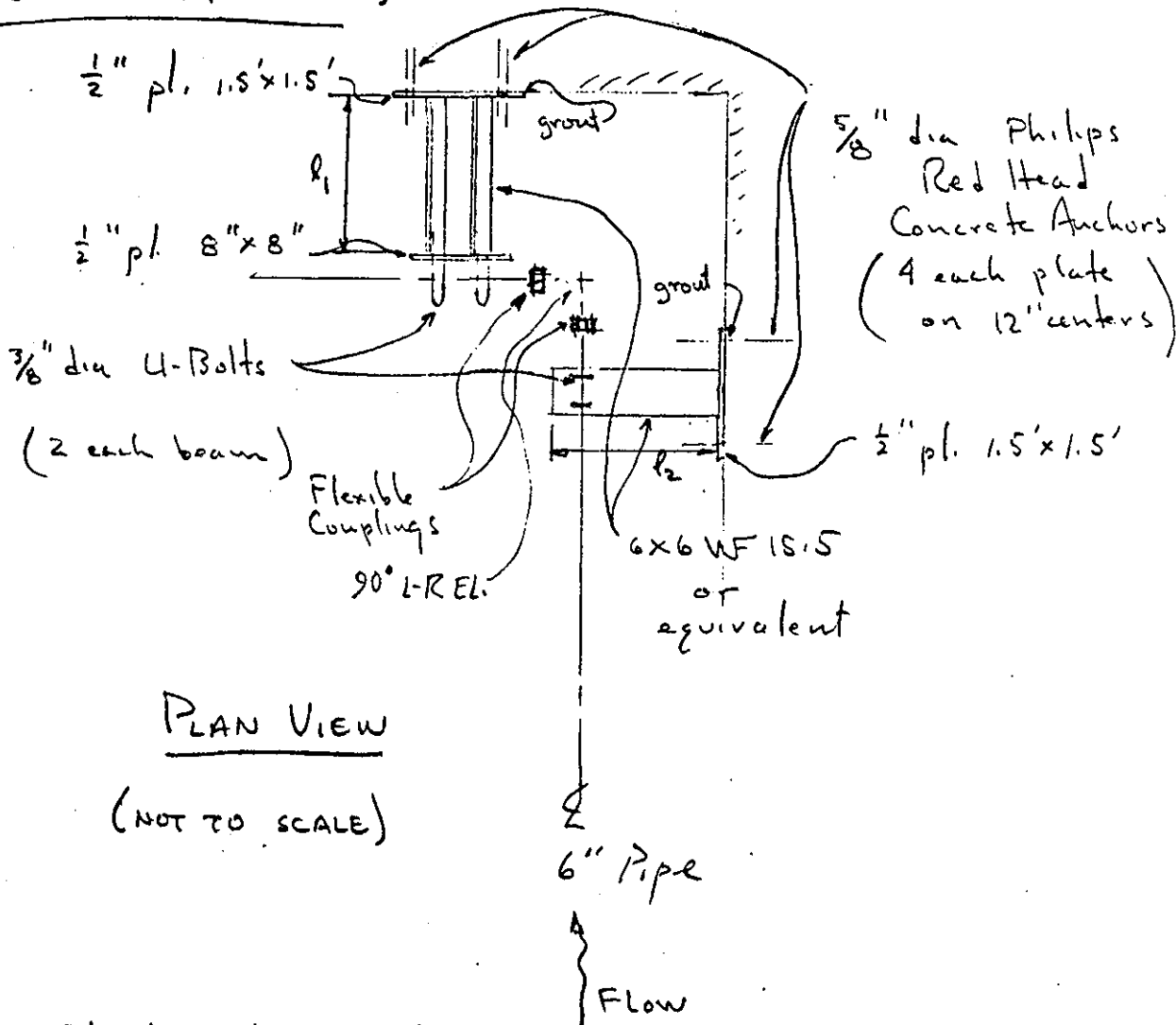
CROSS SECTION THRU 6" PIPE AT

TOWER WALL

DETAIL

IF REQUIRED

Supports for 6" Steel Pipe
(Inside Intake Tower)

PLAN VIEW

(NOT TO SCALE)

Note: ① Steel plates to be welded to WF's with $\frac{1}{2}$ " fillet welds - weld all around.

② h_1 & h_2 as required after tapping of 24" line.

March 31, 1970

INTERDEPARTMENT MAIL

O	File	DEPARTMENT WRC
RO	William H. O'Brien III	DEPARTMENT WRC
SUBJECT Seven Farms Lake Dam - Goshen		

On March 11, 1970 the undersigned and Charles Pelletier, of this office, inspected the subject dam in the company of our consultant, Robert Hoffman, of Machi-Hoffman Engineers, and Mike Taylor, resident engineer for E. D'Appolonia Construction Engineers. This was a semi-final inspection and the work appeared to have progressed satisfactorily to this point. Water was being impounded and was at an approximate 13 foot depth above the stream bed with approximately 13 feet to go to reach the spillway level. There appeared to be no reason why water should not be allowed to be impounded to the spillway level. There were however, the following items which have to be completed before a final inspection:

1. Loaming and seeding of the downstream slope.
2. Installation of stone paved ditches at the toe of the dam leading to the outlet channel.
3. Installation of a permanent locking device on the access cover to the valve well.
4. Installation of valve and stem for low flow augmentation at the southeast end of the primary spillway. There should also be a lock installed on this valve and preferably some sort of a screen arrangement some distance from the opening to eliminate a possible hazard to children when the valve is open.
5. Some sort of weir is to be constructed at the verticle pipe collection point for the toe drains to monitor the seepage through or under the dam, which is collected in these drains, and a report is to be submitted to the Water Resources Commission on this flow.
6. The top of the dam is to be final-graded.

The Water Resources Commission is to be notified when this work has been completed at which time there will be a final inspection.

W. H. O'Brien III

William H. O'Brien III
Civil Engineer

WHO/lch

E. D'APPOLONIA
CONSULTING ENGINEERS, INC.

15 DUFF ROAD
PITTSBURGH, PA. 15235

March 24, 1971

TELEPHONE
(412) 242-5107

Project No. 68-186

Mr. William H. O'Brien, III
Water Resources Commission
State of Connecticut
State Office Building
Hartford, Connecticut 06115

STATE WATER RESOURCES
COMMISSION
RECEIVED

MAR 20 1971

ANSWERED _____
REFERRED _____
FILED _____

Woodridge Lake Dam
(Seven Farms Lake Dam)
Goshen, Connecticut

Dear Mr. O'Brien:

Reference is made to the Woodridge Lake Dam in Goshen, Connecticut and to your letter of April 1, 1970 to Boise Cascade Properties, Inc., wherein you discussed the semi-final inspection of this dam and the items which remain to be constructed. All construction on this dam has been completed and suggested modifications have been incorporated. The reservoir is full and began discharging over the spillway in early March of this year.

We will be pleased to meet with you for a final inspection of this dam as soon as the weather permits. As per our previous discussions at the semi-final inspection, I am enclosing Drawing No. 68-186-B20 showing the piezometric readings during filling of the reservoir. The flow from the outlet of the relief wells has remained relatively constant at about 0.05 cubic feet per second or 150 gallons per hour.

A copy of this letter and all attachments are being sent to Macchi and Hoffman Engineers for their review.

I will look forward to meeting with you at the Woodridge Lake Dam at a time convenient to your schedule.

Very truly yours,

E. D'APPOLONIA CONSULTING ENGINEERS, INC.

Michael J. Taylor
Michael J. Taylor

MJT:tb
Enclosure

cc: Mr. H. Robert Hoffman, Macchi and Hoffman Engineers
Mr. D. Strand, Boise Cascade Recreation Communities Group

September 15, 1971

Mr. Michael J. Taylor
E. D'Appolonia Consulting Engineers, Inc.
10 Duff Road
Pittsburgh, Pennsylvania 15235

Re: Woodbridge Lake Dam
(Seven Farms Lake Dam)
Goshen

Dear Mike:

Thank you for your letter of August 31, 1971 concerning the subject dam with enclosed plans entitled "Platform Details" (drg. No. 68-186-B24) and "Ladder Cage Details" (drg. No. 68-186-B25). By copy of this letter we are forwarding copies of same to our consultant for his comments.

Very truly yours,

William H. O'Brien, III
Civil Engineer

WHO:ljs

MACCHI & HOFFMAN • ENGINEER

EXECUTIVE OFFICES • 44 GILLETT STREET • HARTFORD, CONN., 06105 • PHONE (203) 525-6

A. J. MACCHI, P.E.
H. R. HOFFMAN, P.E.
MICHAEL GIRARD

ASSOCIATE CONSULTANT
PROF. C. W. DUNHAM

July 11, 1972

WATER & RELATE
RESOURCES
RECEIVED

JUL 13 1972

Dept. of Environmental Protection
State of Connecticut
165 Capitol Avenue
Hartford, Connecticut

ANSWERED _____
REFERRED _____
FILED _____

Attention Mr. Wm. H. O'Brien III

Re: Woodridge Lake Dam
Goshen, Conn.

Gentlemen:

On Monday, July 10, 1972 a final check was made on items previously listed as uncompleted.

Present during this inspection at the site were:

Richard DeHahn - Boise Cascade
Michael Taylor - D'Appolonia, Consulting Engineers Inc.
William H. O'Brien and Victor Galgowski - Dept. of Environmental Protection, State of Conn.
A. J. Macchi and J. H. Cosio of Macchi & Hoffman, Engineers

The following specific items were reviewed:

1. In drawdown valve chamber:

- A. A safety cage was installed around steps.
- B. Steel grating platform has been installed over the 36" drain pipe for easy access to valve.
- C. A chain fall has been installed for easy operation of the 6" augmentation valve.

It was noted that water was standing about 3' high in the bottom of the valve chamber almost to the bottom of grating. I commented that the only way to remove this water is with an outside pump. Access to this valve chamber is not a factor in the safety of this facility.

Dept. of Environmental Protection
State of Connecticut
Hartford, Connecticut

July 11, 1972

2. Checked spillway which was found in good condition, overflowing about 2" - 3". A soft spot was found on the west side near bottom. It was concluded that excessive topsoil was dumped here and this prevented proper drainage. M. Taylor will have this corrected by excavating the soft area and replacing with a filter type fill construction.
3. Checked relief well which was flowing clear water measuring about 3" in 90 degree v-notch weir.
4. Toe of dam was checked and no other soft spots were found.

Other than soft spot at spillway which is to be corrected, the dam is found in good condition.

Our letter of September 21, 1971 which recommended certificate of approval is reaffirmed.

Very truly yours,

MACCHI & HOFFMAN, ENGINEERS



A. J. MACCHI



STATE OF CONNECTICUT
DEPARTMENT OF ENVIRONMENTAL PROTECTION
STATE OFFICE BUILDING HARTFORD, CONNECTICUT 06115

WATER RESOURCES

DEC 13 1972

CERTIFICATE OF APPROVAL

Boise-Cascade Properties, Inc.
P.O. Box 66510
Chicago, Illinois 60666

TOWN: Goshen
RIVER: East Branch - Shepaug River
TRIBUTARY: Marshpaug River
CODE NO.:

Gentlemen:

NAME AND LOCATION OF STRUCTURE:

Seven Farms Lake Dam
(Woodbridge Lake Dam)
Marshpaug Road
West Goshen, Connecticut

DESCRIPTION OF STRUCTURE AND WORK PERFORMED:

This is a 1320' long earthen dam with a 80' concrete ogee spillway with a downstream chute creating a pond approximately 390A in area with a maximum depth of 28' immediately above the dam.

CONSTRUCTION PERMIT ISSUED UNDER DATE OF:

September 20, 1966 Renewed September 17, 1968 Revised July 22, 1969

This certifies that the work and construction included in the plans submitted, for the structure described above, has been completed to the satisfaction of this Department and that this structure is hereby approved in accordance with Section 134 of Public Act No. 872.

The owner is required by law to record this Certificate in the land records of the town or towns in which the structure is located.

Dan W. Lufkin
Commissioner

B-64

DWL:WHO:

cc: E. D'Appolonia Consulting Engineers, Inc.

APPENDIX C

DETAIL PHOTOGRAPHS

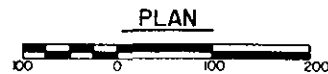
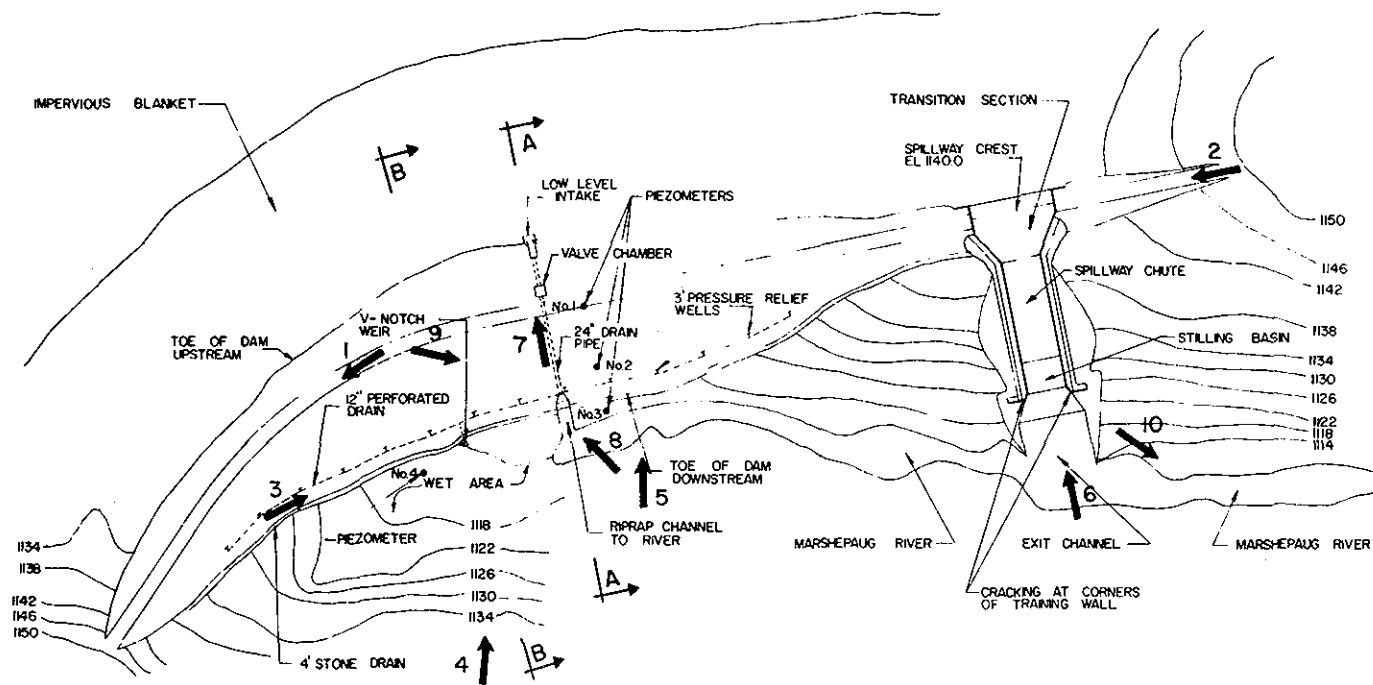


PHOTO LOCATION PLAN

WOODRIDGE LAKE DAM

SHEET C-1



PHOTO 1 - Top and upstream slope riprap of main embankment.



PHOTO 2 - Top and upstream slope of left embankment.

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CAHN ENGINEERS INC.
WALLINGFORD, CONN.
ENGINEER

NATIONAL PROGRAM OF
INSPECTION OF
NON-FED. DAMS

Woodridge Lake Dam
Marshepaug River
Goshen, Connecticut
CE# 27 660 KB
DATE May '79 PAGE C-1



PHOTO 3 - Overgrown stone drain and wet area at right side of dam toe.



PHOTO 4 - Brook from surrounding hills to swamp area at dam toe.

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Woodridge Lake Dam
Marshepaug River
Goshen, Connecticut

CE# 27 660 KB
DATE May '79 PAGE C-2

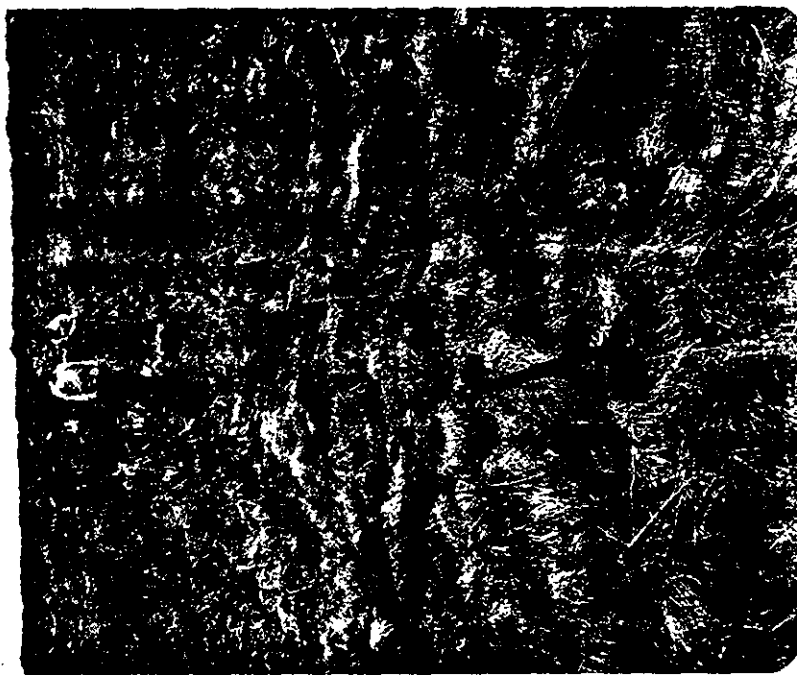


PHOTO 5 - Measuring of water level in piezo-meter #2 and bent pipe of piezo-meter #3.

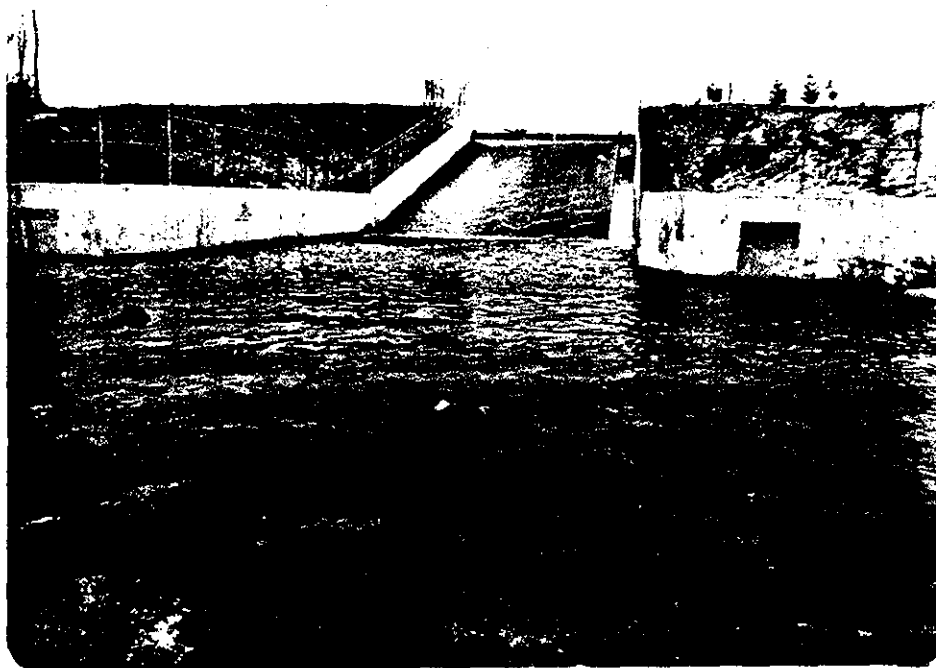


PHOTO 6 - Concrete spillway chute and dumped rock exit channel

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Woodridge Lake Dam
Marshepaug River
Goshen, Connecticut

CE# 27 660 KB

DATE May '79 PAGE C-3



PHOTO 7 - Concrete valve chamber.

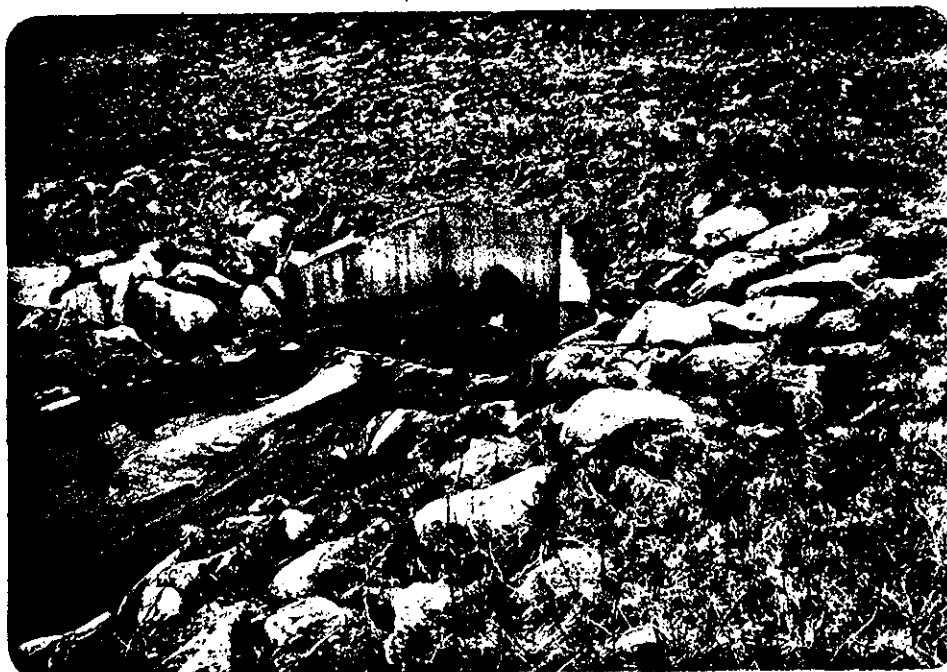


PHOTO 8 - Concrete low level outlet and dumped rock diversion channel.

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Woodridge Lake Dam

Marshepaug River

Goshen, Connecticut

CE# 27 660 KB

DATE May '79 PAGE C-4



PHOTO 9 - Downstream slope. Note outlet at center of photograph.

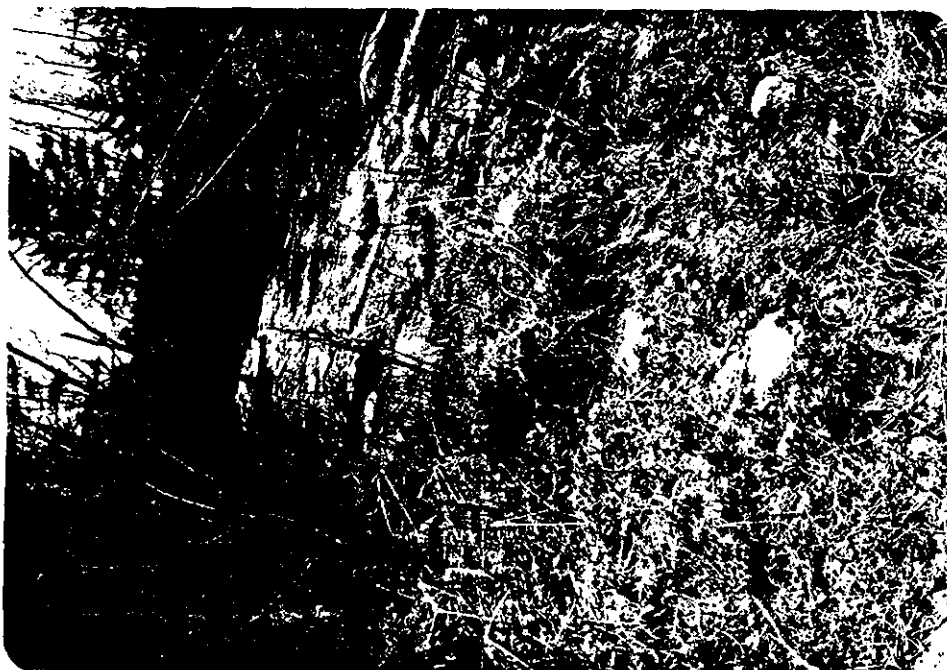


PHOTO 10 - Downstream discharge channel.

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NATIONAL PROGRAM OF
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NON-FED. DAMS

Woodridge Lake Dam

Marshepaug River

Goshen, Connecticut

CE# 27 660 KB

DATE May '79 PAGE C-5

APPENDIX D
HYDRAULICS/HYDROLOGIC COMPUTATIONS

Project INSPECTION OF NON-FEDERAL DAMS IN NEW ENGLAND

Sheet 1 of 12

Computed By RRJ

Checked By J.A. Hill

Date 7/16/79

Field Book Ref. _____

Other Refs. _____

Revisions _____

HYDROLOGIC/HYDRAULIC INSPECTION

WOODBRIDGE LAKE DAM, GOSHEN, CONN.

I) PERFORMANCE AT TEST FLOOD CONDITIONS

1) MAXIMUM PROBABLE FLOOD

a) WATERSHED CLASSIFIED AS "ROLLING"

b) WATERSHED AREA

(i) TOTAL D.A. * 8.89 SQ. MI.

(ii) D.A. U/S FROM TYLER LAKE * 6.44 SQ. MI.

(iii) DIRECT D.A. TO WOODBRIDGE LAKE (D/S FROM TYLER LAKE) 2.45 SQ. MI.

* NOTE: DATA FROM ANDERSON NICHOLS, BOSTON, (REPORT BY JEROME DEGEN) "HYDRAULIC AND HYDROLOGIC CRITERIA FOR DESIGN OF SEVEN FARMS LAKE" MAY 18, 1966; TYLER LAKE, USGS IN HARTFORD, D.A. = 6.42 SQ. MI., NO USGS DRAINAGE RECORD FOR WOODBRIDGE LAKE; CE TYLER LAKE 6.63 SQ. MI., D.A. TO WOODBRIDGE 2.23 SQ. MI.

c) FROM NED-ACE PRELIMINARY GUIDANCE FOR ESTIMATING MAX. PROBABLE DISCHARGES - GUIDE CURVE FOR PMF PEAK FLOW RATES:

(i) PMF \approx 1700 CFS/SQ. MI. FOR TOTAL D.A.

(ii) PMF \approx 1750 CFS/SQ. MI. FOR TYLER LAKE

(iii) PMF \approx 2050 CFS/SQ. MI. FOR DIRECT D.A. TO WOODBRIDGE LAKE

Project NON-FEDERAL DAM INSPECTION

Sheet 2 of 12

Computed By RRJ

Checked By J.A.C. Hu

Date 7/16/79

Field Book Ref. _____

Other Refs. _____

Revisions _____

WOODRIDGE LAKE DAM

I- CONT'D) MAXIMUM PROBABLE FLOOD

d) PEAK INFLOW

BECAUSE A LARGE PORTION OF THE WOODRIDGE LAKE WATERSHED IS REGULATED BY TYLER LAKE (A LAKE OF MODERATE SURFACE AREA AND STORAGE CAPACITY), THE EFFECT OF THIS LAKE ON THE PEAK INFLOW OF WOODRIDGE LAKE SHOULD BE CONSIDERED.

PEAK OUTFLOW AT PMF FOR LAKE TYLER BY APPROXIMATE ROUTING IS ESTIMATED AT $Q_p = 8400$ CFS

SIMILARLY, AT 1/2 PMF, $Q_p' = 3700$ CFS FOR TYLER LAKE

THEREFORE, THE PEAK INFLOW TO WOODRIDGE LAKE IS ESTIMATED AS FOLLOWS:

i) PEAK OUTFLOW FROM TYLER LAKE @ PMF: $(PMF)_1 = Q_p = 8400$ CFS

ii) CONTRIBUTION FROM DIRECT D.A. TO WOODRIDGE LAKE
 $(PMF)_2 = 2.45 \text{ SQ. MI.} \times 1700 \text{ CFS/SQ. MI.} = 4200$ CFS

iii) PEAK INFLOW TO WOODRIDGE LAKE :

$$PMF = 8400 \text{ CFS} + 4200 \text{ CFS} = 12600 \text{ CFS}$$

$$\text{SIMILARLY, "1/2 PMF"} = 3700 \text{ CFS} + 2100 \text{ CFS} = 5800$$

THEREFORE, THE STORAGE EFFECT OF TYLER LAKE REDUCES THE PMF INFLOW OF WOODRIDGE LAKE BY (+) 2500 CFS AND THE 1/2 PMF INFLOW BY (+) 1800 CFS

Project NON-FEDERAL DAM INSPECTIONSheet 3 of 12Computed By R.R.J.Checked By J.A.C. HuxDate 7/16/79

Field Book Ref. _____

Other Refs. _____

Revisions _____

WOODRIDGE LAKE DAM

2) SPILLWAY DESIGN FLOOD

a) CLASSIFICATION OF DAMS ACCORDING TO NED-ACE RECOMMENDED GUIDELINES

i) SIZE*: STORAGE (MAX) 9800 Ac-ft
HEIGHT = 34'

* STORAGE: FROM ANDERSON NICHOLS REPORT MAY 18, 1966 EXHIBIT 17a
DESIGNED STORAGE VS. ELEVATION

ii) HAZARD POTENTIAL: THE DAM IS LOCATED (±) 2 MILES U/S OF THE COMMUNITY OF MILTON (POP ~500). THE CHANNEL BETWEEN WOODRIDGE LAKE DAM AND MILTON PRESENTLY HAS SEVEN RESIDENCES APPROXIMATELY (±) 1/4 MILE D/S AND 10'-12' ABOVE MARSEPAUG RIVER BED WITH EVIDENCE OF NEW DEVELOPMENT TO BEGIN SOON.

iii) CLASSIFICATION

SIZE: INTERMEDIATE

HAZARD: HIGH

b) CSD = PMF = 12600 CFS 1/2 PMF = 5800 CFS

Project WOLF CREEK DAM RECONSTRUCTION

Sheet 5 of 12

Computed By W.A.C. Checked By J.A.C. JH

Date 7/16/79

Field Book Ref. _____ Other Refs. _____

Revisions _____

WOLF CREEK DAM

SL. CONT'D

1. EXTENSION OF RATING CURVE FOR SURCHARGE HEADS ABOVE
TOP OF DAM

THE DAM IS AN EARTH FILL DAM OF 14' TOP WIDTH, 2" TO 1" DIS. FACE SLOPE AND 3" TO 1" DIS. BACK SLOPE. A 24" SATELLITE OUTLET IS PROVIDED THROUGH THE DAM FOR PURPOSES OF DEWATERING AND LOW FLOW RELEASE. THE EMBANKMENT LENGTH EXCLUDING THE SPILLWAY, IS (1) 1245' HORT (TOP EL. 148' MSL) THE TERRAIN TO THE RIGHT OF THE DAM RISES 10' IN A DISTANCE OF (1) 65'; AND TO THE LEFT OF THE DAM RISES 5' IN A DISTANCE OF (2) 70', DESCENDS 3' IN A DISTANCE OF (3) 50' AND THEN RISES 14' IN A DISTANCE OF (4) 180'. BOTH SIDES OF THE DAM ARE WOODED.

ASSUME $C = 3.0$ FOR EARTH EMBANKMENT AND $C = 1.0$ FOR OVERFLOW AT SIDES OF DAM.

ASSUME ALSO, EQUIVALENT LENGTHS FOR THE SPILLWAY, LEFT AND RIGHT EMBANKMENT AND THE SIDES OF THE DAM AS FOLLOWS:

$$L_R = 1/3 (65/10) (H-8) = 4.8 (H-8)$$

$$L_L = 1/3 (70/5) (H-8) = 9.3 (H-8)$$

$$(L_L)_1 = 2/3 (80/4) (H-10) = 13.3 (H-10)$$

$$(L_L)_2 = 2/3 (180/14) (H-10) = 6.5 (H-10)$$

Project NON PERMANENT DAM INSPECTION

Sheet 6 of 12

Computed By R.R.J.

Checked By J.C. Kelly

Date 7/16/79

Field Book Ref. _____

Other Refs. _____

Revisions _____

WOODRIDGE LAKE DAM

3b CON'T) OUTFLOW RATING CURVE

THE TOTAL OUTFLOW RATING CURVE CAN BE APPROXIMATED BY:

$$Q \approx 250 H^{3/2} = 3740 (H-8)^{3/2} + 34 (H-8)^{5/2} + 55 (H-10)^{5/2}$$

C) SPILLWAY CAPACITY TO TOP OF DAM

$$H = 8.0' \quad Q_{SN} \approx 6340 \text{ CFS} \quad ((\pm 5\%) \quad Q_p; \quad (\pm 10\%) \quad Q_p)$$

D) SURCHARGE HEIGHT TO PASS Q_p

$$I) @ Q_p = \text{PMF} = 12500 \text{ CFS} \quad H = 9.3'$$

$$II) @ Q_p = 1/2 \text{ PMF} = 5800 \text{ CFS} \quad H = 7.5'$$

THE OUTFLOW RATING CURVE IS PLOTTED ON THE NEXT PAGE

4) EFFECT OF SURCHARGE ON MAX PROBABLE DISCHARGES (OUTFLOW)

$$a) \text{ LAKE AREA @ FLOW LINE } * A_0 = 385 A_c$$

* FROM ANDERSON-NICHOLS REPORT DATED MAY 1966 (EXHIBIT IVa)
C.F. CHECK MEASURE (1963 1:24000 $A_0 = 379 A_c$ (EL. 1140)
 $A_0 = 422 A_c$ (EL. 1150 MSD))

ASSUME LAKE AREA WITHIN EXPECTED SURCHARGE $A = 400 A_c$

Project NON-FEDERAL DAM INSPECTION

Sheet 7 of 12

Computed By R.R.J.

Checked By E.A.C.

Date 7/16/79

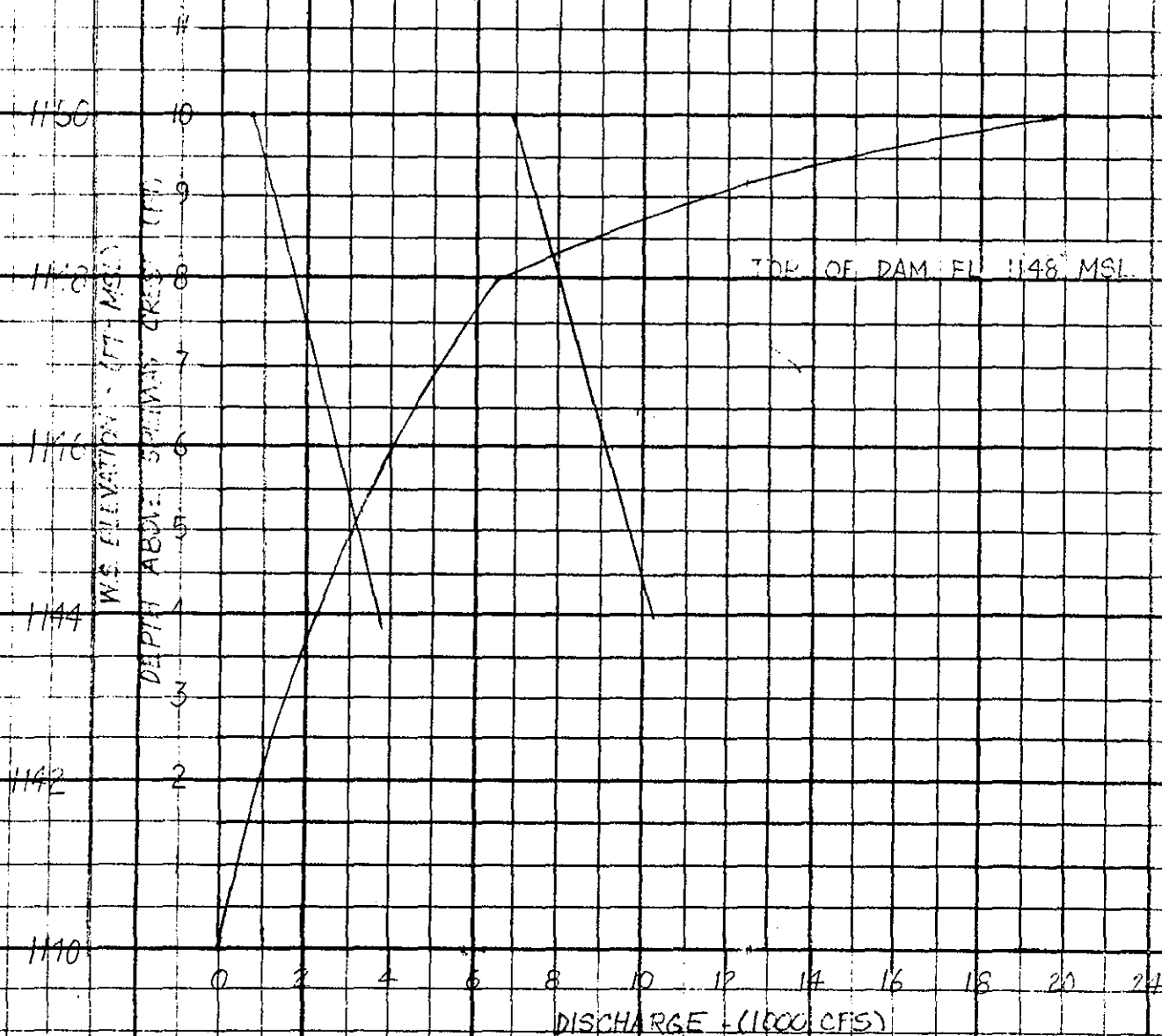
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Other Refs. _____

Revisions _____

WOODRIDGE LAKE DAM

3 - CONT'D) OUTFLOW RATING CURVE



(NOTE : THERE IS A GATED 24" D.I. AT THE BASE OF THE DAM WHICH WOULD RELEASE APPROXIMATELY (+)70 CFS IF OPEN. THIS IS INSIGNIFICANT AS FAR AS Q_p TOTAL, AND WAS NOT FIGURED IN RATING CURVE.)

Project NON FEDERAL DAM INSPECTION

Sheet 8 of 12

Computed By R.R.J.

Checked By HEU

Date 7/16/79

Field Book Ref. _____

Other Refs. _____

Revisions _____

WOODRIDGE LAKE DAM

4-CONT'D) EFFECT OF SURCHARGE STORAGE ON PEAK OUTFLOW

b) ASSUME NORMAL POOL LEVEL AT SPILLWAY CREST (ELEV. 1140' MSL)

c) WATERSHED AREA: D.A. = 8.50 SQ MI

d) DISCHARGE (Q_p) AT VARIOUS SURCHARGE ELEVATIONS

$$H = 10' \quad V = 400 A_c \times 10' = 4000 A_c \cdot Ft \quad S = (4000) / (8.50)(53.3) = 8.44'$$

$$H = 6' \quad V = 400 A_c \times 6' = 2400 A_c \cdot Ft \quad S = (2400) / (8.50)(53.3) = 5.0''$$

FROM APPROXIMATE STORAGE ROUTING GUIDELINES (19" MAX PROBABLE R.O. IN NEW ENGLAND)

$$Q_{p2} = Q_{p1} (1 - S/19) \quad \text{AND FOR 1/2 PMF} \quad Q'_{p2} = Q'_{p1} (1 - S/19.5)$$

$$\therefore \text{FOR } H = 10' \quad Q_{p2} \approx 7000 \text{ CFS} \quad Q'_{p2} = 650 \text{ CFS}$$

$$H = 6' \quad Q_{p2} \approx 9300 \text{ CFS} \quad Q'_{p2} = 2750 \text{ CFS}$$

e) PEAK OUTFLOW (Q_{p3})

USING NED-ACE GUIDELINES "SURCHARGE STORAGE ROUTING"
ALTERNATE METHOD (SEE PG. 7)

$$Q_{p3} = 7600 \text{ CFS} \quad H \approx 8.3' \quad \text{FOR } Q_{p1} = \text{PMF}$$

$$Q'_{p3} = 3200 \text{ CFS} \quad H \approx 5.1' \quad \text{FOR } Q_{p1} = 1/2 \text{ PMF}$$

Project NON FEDERAL DAM INSPECTIONSheet 9 of 12Computed By R.R.J.Checked By L.A.A.Date 7/16/79

Field Book Ref. _____

Other Refs. _____

Revisions _____

WOODRIDGE LAKE DAM

A-CONT'D) EFFECT OF SURCHARGE STORAGE ON PEAK OUTFLOW

F) SPILLWAY CAPACITY TO OUTFLOW

SPILLWAY CAPACITY TO TOP OF DAM $Q_s \approx 6340$ CFS \therefore SPILLWAY CAPACITY IS $(\pm) 81\%$ THE OUTFLOW @ PMF AND $(\pm) 128\%$ THE OUTFLOW AT $1/2$ PMF

5) SUMMARY

a) PEAK INFLOW $Q_p \approx 12600$ CFS $Q_p' = 1/2$ PMF = 5800 CFSb) PEAK OUTFLOW $Q_b \approx 7850$ CFS $Q_b' = 3200$ CFSc) SPILLWAY MAX CAPACITY $Q_s = 6340$ CFS OR $(\pm) 81\%$ OF Q_b AND 128% OF Q_b' THEREFORE, AT SDF = PMF, THE DAM IS OVERTOPPED $(\pm) 0.3'$ (WS EL 1148' MSL) OR TO AN AVERAGE SURCHARGE ABOVE THE SPILLWAY CREST OF $(\pm) 8.3'$ AT SDF = $1/2$ PMF, THE SPILLWAY HANDLES THE ENTIRE FLOW, UTILIZING A LITTLE MORE THAN 50% OF ITS CAPACITY (WS EL 1145.1' MSL) WITH AN AVERAGE SURCHARGE ABOVE THE SPILLWAY CREST OF $(\pm) 5.1'$

Project NON-FEDERAL DAMS INSPECTIONSheet 10 of 12Computed By RRJChecked By J.A.C.Date 7/16/79

Field Book Ref. _____

Other Refs. _____

Revisions _____

WOODRIDGE LAKE DAM

II) DOWNSTREAM FAILURE HAZARD

1) PEAK FLOOD AND STAGE IMMEDIATELY DOWNSTREAM FROM DAM

a) BREACH WIDTH

1) MID-HEIGHT (\pm) ELEV 1131 MSL ($1148 - 33.5/2 = 1131.25$, SAY 1131.0)2) APPROX. MID-HEIGHT LENGTH $L \approx 712'$ (C.E. FROM A&N DRAWING)

3) BREACH WIDTH (SEE NED / ACE D/S DAM FAILURE GUIDELINES):

$$W = 0.4 \times 712 = 284.8 \quad \therefore \text{ASSUME } W_b = 285'$$

b) PEAK FAILURE OUTFLOW (Q_p) (SEE NOTE PAGE : 7)

ASSUME SURCHARGE TO TOP OF DAM: THEREFORE

1) HEIGHT AT TIME OF FAILURE $h_o = 33.5'$ 2) SPILLWAY DISCHARGE $Q_s = 6300 \text{ CFS}$

BECAUSE OF THE HIGH TAILWATER (WS EL. 1121.0) EXISTING JUST BEFORE FAILURE, THE ACTUAL HEAD (h_o) WHICH WILL PRODUCE THE PEAK FAILURE OUTFLOW WILL BE APPROXIMATELY

$$h_o = 25'$$

* FROM ANDERSON-NICHOLS, MASS. DNG "SEVEN FARMS LAKE - DAM PLAN AND SECTIONS", DATED JULY 1966. LOWEST ELEV. OF D/S TOE OF DAM 1114.5 MSL, TOP OF DAM ELEV 1148 MSL, \therefore HEIGHT (MAX) $\approx 33.5'$

Project NON FEDERAL DAM INSPECTIONSheet 11 of 12Computed By R.R.J.Checked By JAC. JLLDate 7/16/79

Field Book Ref. _____

Other Refs. _____

Revisions _____

WOODRIDGE LAKE DAM

1b- CONT'D) PEAK FAILURE OUTFLOW

III) BREACH OUTFLOW (Q_b)

$$Q_b = (8/27) W_b \sqrt{g} h_o^{3/2} \approx 60000 \text{ CFS}$$

$$\text{IV) PEAK FAILURE OUTFLOW } (Q_p) = Q_s + Q_b = 6300 + 60000 \approx 66300 \text{ CFS}$$

C) RAISE IN STAGE ABOVE TAILWATER IMMEDIATELY D/S FROM DAM

$$h \approx 0.44 h_o \approx 11.0'$$

d) APPROXIMATE STAGE JUST BEFORE FAILURE

$$i) Q = Q_s \approx 6300 \text{ CFS}$$

THE CHANNEL JUST D/S FROM THE DAM SLOPES APPROXIMATELY 0.0080, DROPPING (\pm) 10^V IN A DISTANCE OF (\pm) 1300^H . THE TERRAIN SLOPES APPROXIMATELY 10^H TO 1^V TO THE RIGHT OF THE CHANNEL AND 1^V TO 15^H TO THE LEFT.

$$ii) \text{ STAGE FOR } Q_s \quad Y_s \approx 8.5' \text{ FOR } Q_s \approx 6300 \text{ CFS}$$

e) FLOOD STAGE AFTER FAILURE AT CHANNEL (\pm) 1300^H D/S FROM DAM (IMMEDIATE IMPACT AREA)

$$Y_p \approx 20.5' \text{ FOR } Q_p \approx 66300 \text{ CFS}$$

$$f) \text{ RAISE IN STAGE IN IMMEDIATE IMPACT AREA } \Delta Y = Y_p - Y_s \approx 12.0'$$

Project NON-FEDERAL DAM INSPECTIONSheet 12 of 12Computed By R.R.J.Checked By JACDate 7/16/74

Field Book Ref. _____

Other Refs. _____

Revisions _____

WOODRIDGE LAKE DAM

2) SUMMARY

a) PEAK FAILURE OUTFLOW

$$Q_p \approx 66300 \text{ CFS}$$

b) RAISE IN STAGE JUST D/S FROM DAM

$$h = 0.44 h_0 \approx 11.0'$$

c) APPROXIMATE STAGE BEFORE FAILURE

$$Y_s \approx 8.5'$$

d) APPROXIMATE STAGE AFTER FAILURE AT IMMEDIATE IMPACT AREA

$$Y_p \approx 20.5'$$

e) RAISE IN STAGE AT IMMEDIATE IMPACT AREA

$$\Delta Y \approx 20.5 - 8.5 \approx 12.0'$$

PRELIMINARY GUIDANCE
FOR ESTIMATING
MAXIMUM PROBABLE DISCHARGES
IN
PHASE I DAM SAFETY
INVESTIGATIONS

New England Division
Corps of Engineers

March 1978

MAXIMUM PROBABLE FLOOD INFLOWS
NED RESERVOIRS

<u>Project</u>	<u>Q</u> (cfs)	<u>D.A.</u> (sq. mi.)	<u>MPF</u> cfs/sq. mi.
1. Hall Meadow Brook	26,600	17.2	1,546
2. East Branch	15,500	9.25	1,675
3. Thomaston	158,000	97.2	1,625
4. Northfield Brook	9,000	5.7	1,580
5. Black Rock	35,000	20.4	1,715
6. Hancock Brook	20,700	12.0	1,725
7. Hop Brook	26,400	16.4	1,610
8. Tully	47,000	50.0	940
9. Barre Falls	61,000	55.0	1,109
10. Conant Brook	11,900	7.8	1,525
11. Knightville	160,000	162.0	987
12. Littleville	98,000	52.3	1,870
13. Colebrook River	165,000	118.0	1,400
14. Mad River	30,000	18.2	1,650
15. Sucker Brook	6,500	3.43	1,895
16. Union Village	110,000	126.0	873
17. North Hartland	199,000	220.0	904
18. North Springfield	157,000	158.0	994
19. Ball Mountain	190,000	172.0	1,105
20. Townshend	228,000	106.0(278 total)	820
21. Surry Mountain	63,000	100.0	630
22. Otter Brook	45,000	47.0	957
23. Birch Hill	88,500	175.0	505
24. East Brimfield	73,900	67.5	1,095
25. Westville	38,400	99.5(32 net)	1,200
26. West Thompson	85,000	173.5(74 net)	1,150
27. Hodges Village	35,600	31.1	1,145
28. Buffumville	36,500	26.5	1,377
29. Mansfield Hollow	125,000	159.0	786
30. West Hill	26,000	28.0	928
31. Franklin Falls	210,000	1000.0	210
32. Blackwater	66,500	128.0	520
33. Hopkinton	135,000	426.0	316
34. Everett	68,000	64.0	1,062
35. MacDowell	36,300	44.0	825

MAXIMUM PROBABLE FLOWS
BASED ON TWICE THE
STANDARD PROJECT FLOOD
(Flat and Coastal Areas)

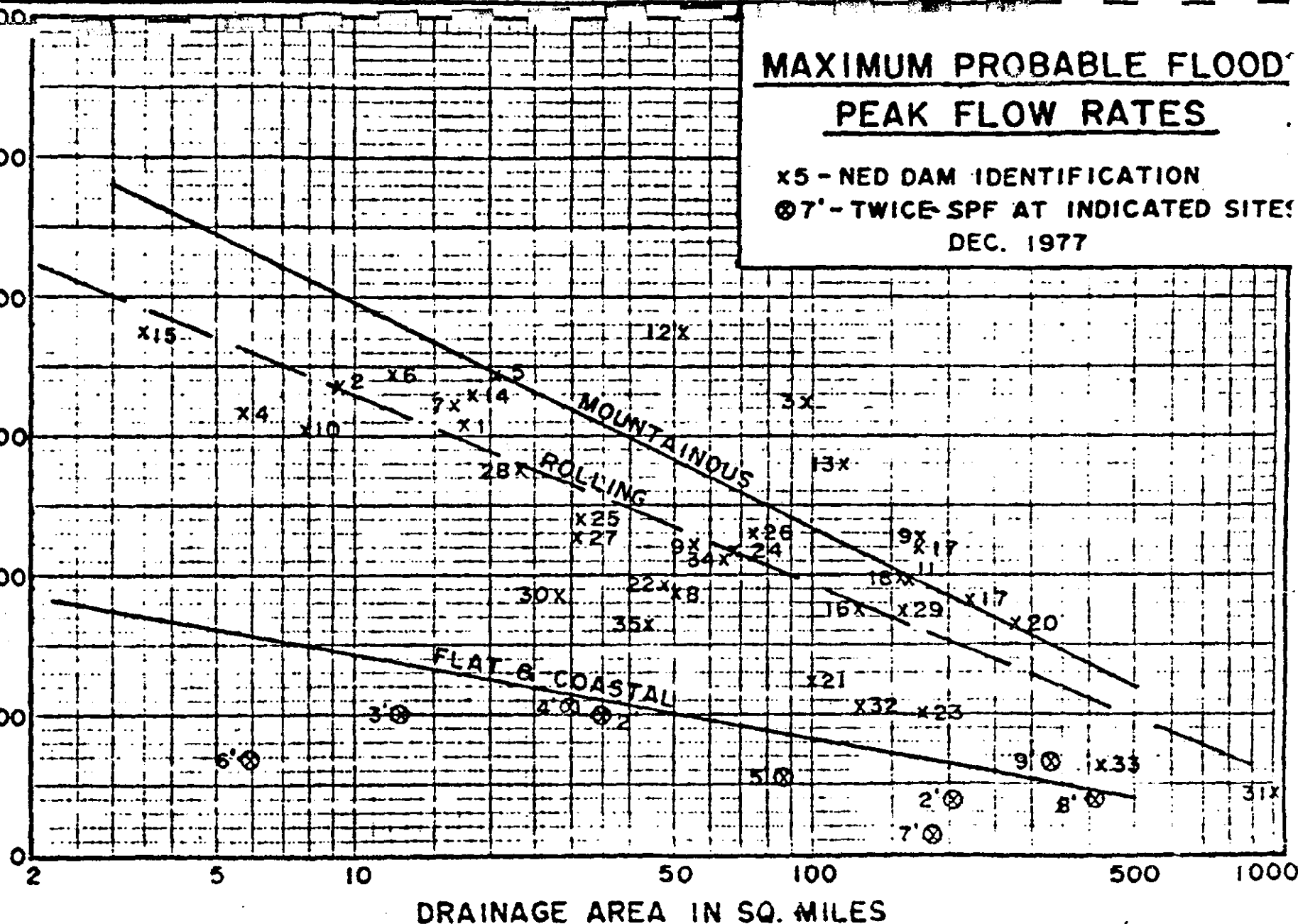
<u>River</u>	<u>SPF</u> (cfs)	<u>D.A.</u> (sq. mi.)	<u>MPF</u> (cfs/sq. mi.)
1. Pawtuxet River	19,000	200	190
2. Mill River (R.I.)	8,500	34	500
3. Peters River (R.I.)	3,200	13	490
4. Kettle Brook	8,000	30	530
5. Sudbury River.	11,700	86	270
6. Indian Brook (Hopk.)	1,000	5.9	340
7. Charles River.	6,000	184	65
8. Blackstone River.	43,000	416	200
9. Quinebaug River	55,000	331	330

AT

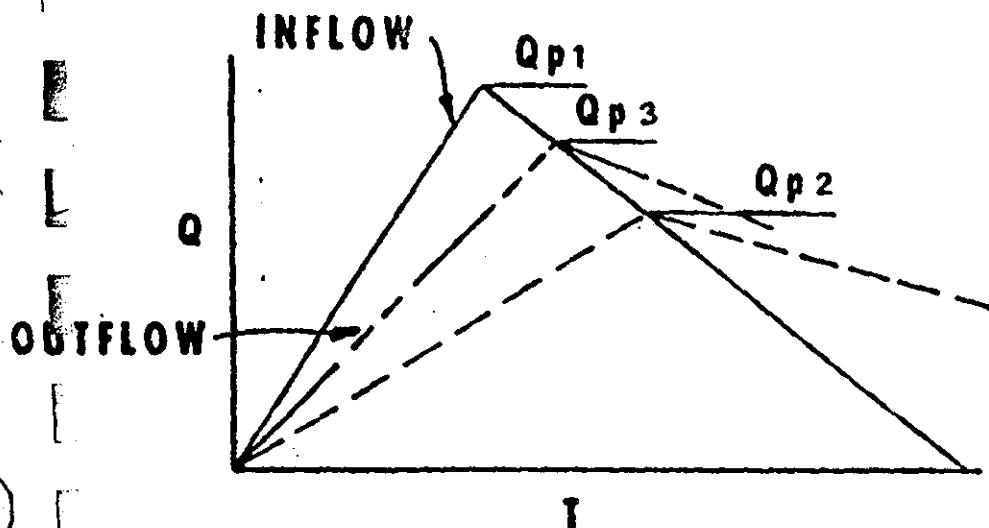
M.P.F. IN C.F.S./SQ. MILE

MAXIMUM PROBABLE FLOOD PEAK FLOW RATES

x5 - NED DAM IDENTIFICATION
⊗7' - TWICE SPF AT INDICATED SITES
DEC. 1977



ESTIMATING EFFECT OF SURCHARGE STORAGE ON MAXIMUM PROBABLE DISCHARGES



STEP 1: Determine Peak Inflow (Q_{p1}) from Guide Curves.

STEP 2: a. Determine Surcharge Height To Pass " Q_{p1} ".

b. Determine Volume of Surcharge ($STOR_1$) In Inches of Runoff.

c. Maximum Probable Flood Runoff In New England equals Approx. 19", Therefore

$$Q_{p2} = Q_{p1} \times \left(1 - \frac{STOR_1}{19}\right)$$

STEP 3: a. Determine Surcharge Height and " $STOR_2$ " To Pass " Q_{p2} "

b. Average " $STOR_1$ " and " $STOR_2$ " and Determine Average Surcharge and Resulting Peak Outflow " Q_{p3} ".

SURCHARGE STORAGE ROUTING SUPPLEMENT

**STEP 3: a. Determine Surcharge Height and
"STOR₂" To Pass "Q_{p2}"**

**b. Avg "STOR₁" and "STOR₂" and
Compute "Q_{p3}".**

**c. If Surcharge Height for Q_{p3} and
"STOR_{AVG}" agree O.K. If Not:**

**STEP 4: a. Determine Surcharge Height and
"STOR₃" To Pass "Q_{p3}"**

**b. Avg. "Old STOR_{AVG}" and "STOR₃"
and Compute "Q_{p4}"**

**c. Surcharge Height for Q_{p4} and
"New STOR_{AVG}" should Agree
closely**

SURCHARGE STORAGE ROUTING ALTERNATE

$$Q_{p2} = Q_{p1} \times \left(1 - \frac{\text{STOR}}{19} \right)$$

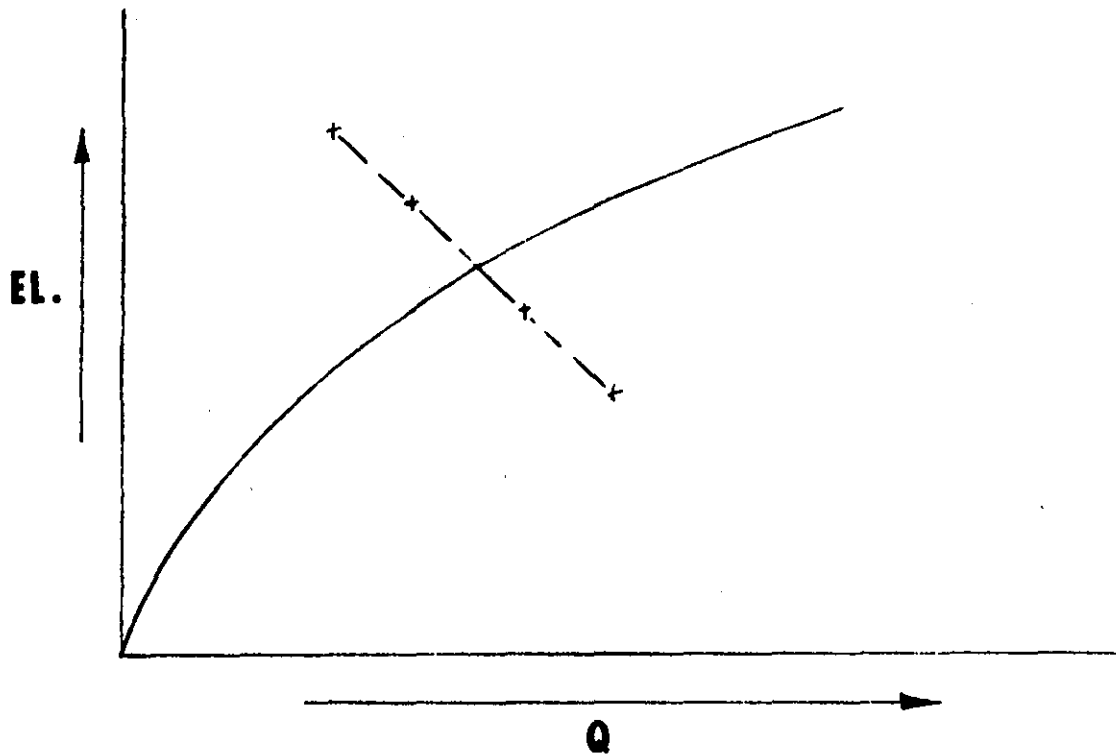
$$Q_{p2} = Q_{p1} - Q_{p1} \left(\frac{\text{STOR}}{19} \right)$$

FOR KNOWN Q_{p1} AND 19" R.O.

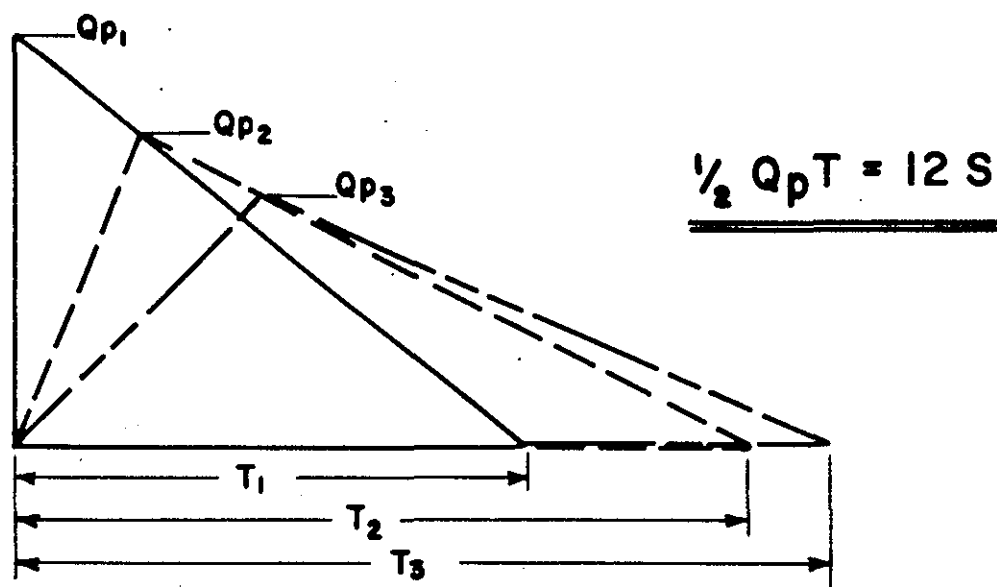
Q_{p2}

STOR

EL.



"RULE OF THUMB" GUIDANCE FOR ESTIMATING DOWNSTREAM DAM FAILURE HYDROGRAPHS



STEP 1: DETERMINE OR ESTIMATE RESERVOIR STORAGE (S) IN AC-FT AT TIME OF FAILURE.

STEP 2: DETERMINE PEAK FAILURE OUTFLOW (Q_{p1}).

$$Q_{p1} = \frac{8}{27} W_b \sqrt{g} Y_0^{3/2}$$

W_b = BREACH WIDTH - SUGGEST VALUE NOT GREATER THAN 40% OF DAM LENGTH ACROSS RIVER AT MID HEIGHT.

Y_0 = TOTAL HEIGHT FROM RIVER BED TO POOL LEVEL AT FAILURE.

STEP 3: USING USGS TOPO OR OTHER DATA, DEVELOP REPRESENTATIVE STAGE-DISCHARGE RATING FOR SELECTED DOWNSTREAM RIVER REACH.

STEP 4: ESTIMATE REACH OUTFLOW (Q_{p2}) USING FOLLOWING ITERATION.

A. APPLY Q_{p1} TO STAGE RATING, DETERMINE STAGE AND ACCOMPANYING VOLUME (V_1) IN REACH IN AC-FT. (NOTE: IF V_1 EXCEEDS $1/2$ OF S, SELECT SHORTER REACH.)

B. DETERMINE TRIAL Q_{p2} .

$$Q_{p2}(\text{TRIAL}) = Q_{p1} \left(1 - \frac{V_1}{S}\right)$$

C. COMPUTE V_2 USING Q_{p2} (TRIAL).

D. AVERAGE V_1 AND V_2 AND COMPUTE Q_{p2} .

$$Q_{p2} = Q_{p1} \left(1 - \frac{V_{\text{avg}}}{S}\right)$$

STEP 5: FOR SUCCEEDING REACHES REPEAT STEPS 3 AND 4.

APRIL 1978

APPENDIX E

INFORMATION AS CONTAINED IN
THE NATIONAL INVENTORY OF DAMS



INVENTORY OF DAMS IN THE UNITED STATES

STATE	IDENTITY NUMBER	DIVISION	STATE	COUNTY	CONGR DIST.	STATE	COUNTY	CONGR DIST.	NAME	LATITUDE (NORTH)	LONGITUDE (WEST)	REPORT DATE DAY MO YR
CT	452	NED	CT	005	06				WOODRIDGE LAKE DAM	4147.8	7315.1	31AUG79

POPULAR NAME	NAME OF IMPOUNDMENT
	WOODRIDGE LAKE

REGION	BASIN	RIVER OR STREAM	NEAREST DOWNSTREAM CITY-TOWN-VILLAGE	DIST FROM DAM (MI.)	POPULATION
01	07	MARSHEPAUG	MILTON	2	500

TYPE OF DAM	YEAR COMPLETED	PURPOSES	STRUC- HEIGHT (FT.)	HYDRAU- HEIGHT (FT.)	IMPOUNDING CAPACITIES	
					MAXIMUM (ACRE-FT.)	NORMAL (ACRE-FT.)
REPG	1970	R	34	34	9800	6500

DIST OWN FED R PRV/FED SCS A VER/DATE

NED N N N N

REMARKS

(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)
D/S HAS	SPILLWAY			MAXIMUM DISCHARGE (FT.)	VOLUME OF DAM (CY)	POWER CAPACITY		NAVIGATION LOCKS										
	CHEST LENGTH	TYPE	WIDTH (FT.)			INSTALLED (MW)	PROPOSED (MW)	NO	LENGTH (FT.)	WIDTH (FT.)	LENGTH (FT.)	WIDTH (FT.)	LENGTH (FT.)	WIDTH (FT.)	LENGTH (FT.)	WIDTH (FT.)		
1	1320	U	80	6340														

OWNER	ENGINEERING BY	CONSTRUCTION BY
WOODRIDGE LAKE ASSOC.	ANDERSON-NICHOLS INC.	E D'APPOLONIA

REGULATORY AGENCY			
DESIGN	CONSTRUCTION	OPERATION	MAINTENANCE
CT WATER RESOURCES	CT WATER RESOURCES	CT WATER RESOURCES	CT WATER RESOURCES

INSPECTION BY	INSPECTION DATE			AUTHORITY FOR INSPECTION
	DAY	MO	YR	
CAHN ENGINEERS INC	03	MAY	79	PL 92-367

REMARKS
47-ALSO E D'APPOLONIA ENGINEERS